Plate-girder Railway Bridges

Maurice Fitzmaurice

PLATE-GIRDER RAILWAY BRIDGES

BY

MAURICE FITZMAURICE

B.A., B.E., M. INST. C.E.

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PREFACE.

THE great majority of bridges on a line of railway are short-span plate-girder bridges. This book is intended to help those young engineers who, joining an engineer's office for the first time, find themselves engaged in this class of work. All complicated calculations have been avoided and the object in view has been to help the student or pupil to design a bridge of this kind efficiently, cheaply, simply, and in accordance with modern practice.

The theoretical knowledge required is small, and an attempt has been made not to enlarge on this part of the subject more than absolutely necessary.

Practical details with regard to construction and erection, together with particulars of market sizes of plates, &c., have been given as far as possible.

A chapter has been given to specifications for steel and iron, and to some notes on the treatment of these metals in the manufacture of bridge work, accompanied by some remarks on the working stresses to be adopted in these materials.

The detailed drawings of bridges, and the calculations accompanying them will, it is probable, be more useful than abstract remarks on the subject. Several kinds of modern trough floors have been given, with remarks thereon.

While space has not allowed detailed reference to more than a few types of bridges, it is hoped that enough has been said, with regard to theoretical principles and practical details, to simplify the design of any structure which may be presented under the head of Plate-girder Railway Bridges.

M. F.

June 1895.

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PLATE-GIRDER RAILWAY BRIDGES.

CHAPTER I.

STRAINS IN BEAMS AND GIRDERS.

WE shall assume for the present that all the external forces acting on a girder are vertical.

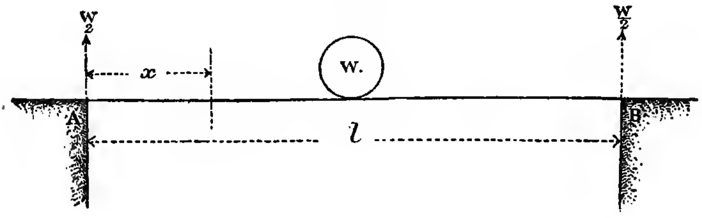
Consider the external forces on one side of any vertical section of a girder, and take the moment of each of these forces around this section; the sum of all these moments is called the *bending moment* at this section.

Consider the case of a girder supported at both ends and loaded in the centre with a concentrated load as in Fig. 1. We shall call the weight in the middle W. This weight is supported by the girder A B, which in turn is supported by the two abutments. It is evident that the reaction at each abutment is equal to $\frac{W}{2}$. The bending moment at the centre is, from our definition,

$$\frac{W}{2} \times \frac{l}{2} = \frac{W\,l}{4}.$$

Let us now take a vertical section distant x from the left abutment. Considering the forces to the left of this section, we find the only force is the

FIG. I.



reaction at the left abutment which is $\frac{W}{2}$; the bending moment at the section we have taken is therefore $\frac{W}{2} \times x$. If we consider the forces to the right of the section we have the force W and the reaction $\frac{W}{2}$, and the bending moment at the section due to these forces is,

$$\frac{W}{2}(l - x) - W\left(\frac{l}{2} - x\right) = \frac{W}{2}l - \frac{W}{2}x - \frac{W}{2}l$$

$$+ Wx = \frac{W}{2} \times x,$$

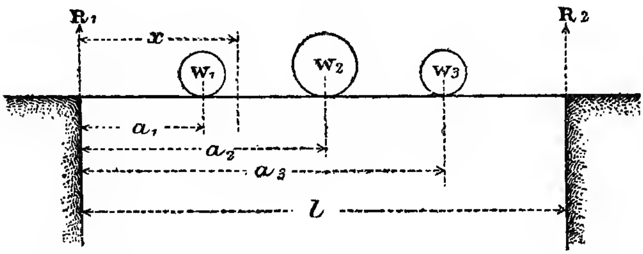
which, of course, is the same as we obtained by considering the forces on the left side of the section.

We shall now consider the girder supported at both ends as before, but having more than one weight

on it. We shall call the left reaction R_1 and the right reaction R_2.

Fig. 2 shows the girder with three weights W_1, W_2 and W_3 on it, at the distances shown from the left abutment.

FIG. 2.



We know that the reactions are,

$$R_1 = W_1 \frac{l - a_1}{l} + W_2 \frac{l - a_2}{l} + W_3 \frac{l - a_3}{l},$$

$$R_2 = W_1 \frac{a_1}{l} + W_2 \frac{a_2}{l} + W_3 \frac{a_3}{l}.$$

Consider the bending moment at a section distant x from the left abutment. The forces to the left of this section are R_1 and W_1. The bending moment at section is therefore,

$$R_1 \, x - W_1 \, (x - a_1).$$

If we now consider the forces to the right of the section we find R_2, W_2 and W_3, and the bending moment at the section is,

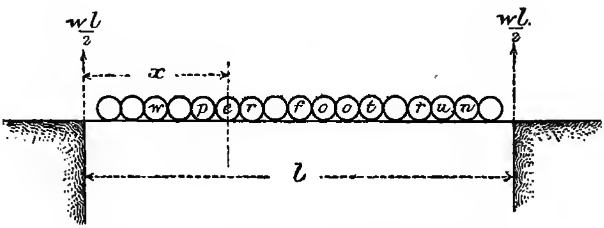
$$R_2 \, (l - x) - W_3 \, (a_3 - x) - W_2 \, (a_2 - x).$$

If, in these two expressions for the bending moment at this section, we substitute for R_1 and R_2 their values in terms of the weights, we shall find that the expressions are of course identical.

In a similar way the bending moment at any other section can be found.

Fig. 3 represents a girder supported at both ends, and uniformly loaded with w tons per foot run. It is evident that the reaction at each abutment is $\frac{w\,l}{2}$.

FIG. 3.



Take a vertical section at a distance x from the left abutment. The forces to the left of this section are the reaction $\frac{w\,l}{2}$, distance x from the section and the load $w\,x$, whose centre of gravity is distant $\frac{x}{2}$ from the section. The bending moment at this section is therefore,

$$\frac{w\,l}{2} \times x - w\,x \times \frac{x}{2} = \frac{w\,l\,x}{2} - \frac{w\,x^2}{2}.$$

If we consider the forces at the right-hand side of the same section we have the reaction $\frac{w\,l}{2}$ and the

weight $(l - x)\ w$. The bending moment at the section due to these weights is therefore,

$$\frac{w\ l}{2} \times (l - x) - (l - x)\ w \times \frac{l - x}{2}$$

$$= (l - x)\left(\frac{w\ l}{2} - \frac{w\ (l - x)}{2}\right) = \frac{w\ l}{2} x - \frac{w\ x^2}{2},$$

which is the same as we have already got by considering the weights on the left hand.

The bending moment at the centre is, similarly,

$$\frac{w\ l}{2} \times \frac{l}{2} - \frac{w\ l}{2} \times \frac{l}{4} = \frac{w\ l^2}{4} - \frac{w\ l^2}{8} = \frac{w\ l^2}{8}.$$

If we represent the total load $w\ l$ by W, the bending moment at the centre is $\frac{\mathrm{W}\ l}{8}$.

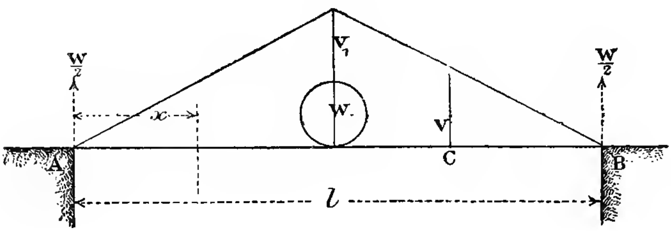
We see, therefore, that the bending moment at the centre of a girder, with a uniformly distributed load, is half the bending moment at the centre of the same girder, if the same load is concentrated at the centre as in Fig. 1.

Referring back now to Fig. 1: It is evident that the bending moment at any section varies directly as its distance from the abutment. We therefore erect at the centre a vertical equal on any scale to the bending moment at the centre, and join the extremity of this vertical with the points of support; the ordinates at any other section on the same scale will represent the bending moment at that section.

In Fig. 4 the vertical V_1 at the centre represents

the bending moment at the centre on some particular scale (say a certain number of foot-tons to the inch). The extremity of this vertical is joined to the points

FIG. 4.



of support, then the ordinate V, at any section C, is the bending moment at that section to the same scale.

Let us now consider Fig. 2. If we calculate the values of the bending moments at each weight, and set up at each weight verticals equal to these bending moments, and join their extremities, and also the extremities of the end ones with the points of support, the ordinates thus given will represent the bending moments at their respective points.

In Fig. 5, V_1 V_2 V_3 are the verticals set up at the weights W_1 W_2 and W_3, equal on some scale to the bending moments at these points. Then the bending moment at any other point C is represented by the ordinate V.

It may not be quite clear that the line joining the extremities of the verticals, which represent on a certain scale the bending moments at the points where the weights are situated, will give the bending

moments at the intermediate points, but this can easily be made clear.

The bending moment at a point distant x from the left abutment, and situated between the weights W_1 and W_2, we have found to be,

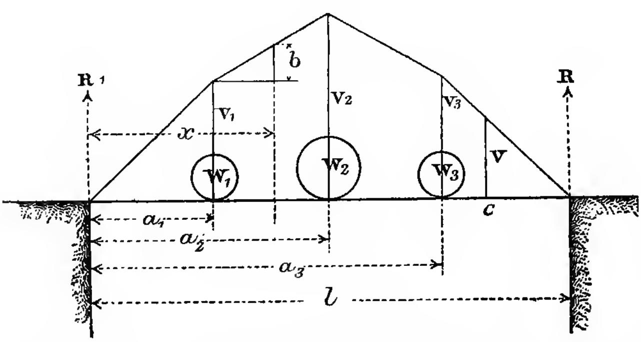
$$R_1 x - W_1 (x - a_1),$$

and the bending moment at the weights W_1 is $R_1 a_1$. The difference between these bending moments is,

$$R_1 x - W_1 (x - a_1) - R_1 a_1 = R_1 (x - a_1) - W_1 (x - a_1)$$
$$= (R_1 - W_1)(x - a_1);$$

therefore, the increase of bending moment shown by length b on diagram between the weight W_1 and any point situated between W_1 and W_2, any distance x from the left abutment, varies directly as $(x - a_1)$; that is, it varies directly with its distance from W_1;

FIG. 5.

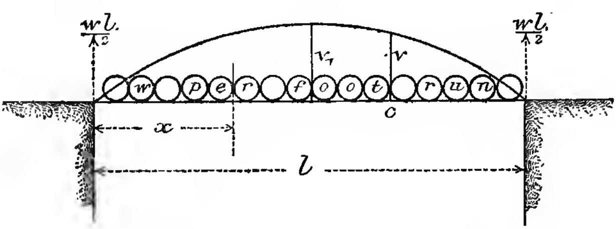


and the upper extremity of the lengths corresponding to these differences must lie on a line passing through the upper extremity of the vertical at W_1, and we

know that at the weight W_2 this line must pass through the extremity of the vertical there; therefore the ordinates of the line joining the extremities of the verticals erected at W_1 and W_2, and which represent the bending moments at these points, must give the bending moments at all intermediate points to the same scale, and similarly, it can be proved between any other verticals.

Referring now to Fig. 3: From the equation of the bending moment we can see that the line joining the extremities of the bending moment ordinates is a parabola. If we therefore erect at centre a vertical V_1 equal to the bending moment at the centre on any scale, and describe a parabola having its vertex at the extremity of V_1, and passing through the points of support, the ordinate at any point will give the bending moment at that point on the same scale. This is done in Fig. 6. For instance, at the

FIG. 6.



point C the ordinate V gives the bending moment at that point. When the ordinate in the centre is taken on such a scale so that V_1 is not more than one-tenth of l, a circle and parabola are for practical

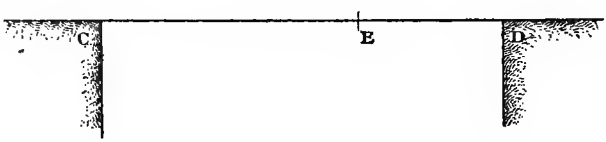
purposes the same, and the former may be described instead of the latter.

When in addition to a uniform load on a girder one or more isolated loads are placed on it the bending moment at any point will be of course the sum of the bending moments at that point for each kind of loading.

For a given intensity of load per unit of length, a uniform load over the whole girder produces a greater bending moment at each cross section than any partial load.

"Let the two ends of the girder be called C and D, and any intermediate cross-section E. Then for a *uniform* load the bending moment at E is an

FIG. 7.



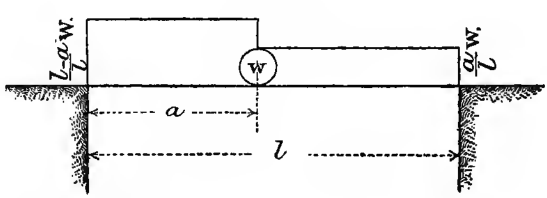
upward moment, being equal to the upward moment of the supporting forces at either of the ends relatively to E *minus* the downward moment of the uniform load between that end and E. A *partial* load is produced by removing the uniform load from part of the beam situated either between E and C, between E and D, or at both sides of E. First let the load be removed from any part of the beam between E and C; then the downward moment, relatively to E, of the load between E and D is unal-

tered ; and the upward moment, relatively to E, of the supporting force at D, is diminished in consequence of the diminution of that force ; therefore the bending moment at E is diminished. Similarly, it can be proved that if the load be removed from part of the girder between E and D, the bending moment at E is diminished, and the combined effect of these operations takes place when the load is removed from portions of the beam lying on both sides of E ; so that the removal of the load from any portion of the beam diminishes the bending moment at each point." *

Before considering how a girder resists the stresses produced by the bending moments at the different sections we shall first shortly consider what are called the *shearing stresses* in girders.

The shearing strain at any section of a girder is the amount of the load transmitted through that section to the abutment.

FIG. 8.



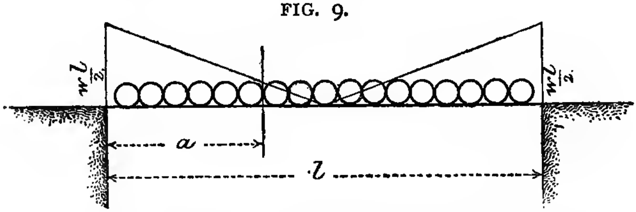
In Fig. 8 we have a girder loaded with a weight W at a distance *a* from the left abutment. The amount of the load transmitted to the left abutment

* Rankine, 'Applied Mechanics.'

is $\frac{l-a}{l}$ W, and to the right abutment is $\frac{a}{l}$ W. The shearing stress therefore at any point between W and the left abutment is $\frac{l-a}{l}$ W, and between W and the right abutment is $\frac{a}{l}$ W. If the weight were in the centre of the girder the shearing strain at any point between the centre and the abutment would be $\frac{W}{2}$.

In Fig. 9 we have a girder loaded uniformly with a weight of w per unit of length. The amount of weight transmitted to each abutment is $\frac{w\,l}{2}$.

FIG. 9.



Consider a section distant a from the left abutment. The amount of the load transmitted through this section to the abutment is evidently the reaction of the abutment, less the weight between the section and the abutment, and is therefore $\frac{w\,l}{2} - w\,a$ or $w\left(\frac{l}{2} - a\right)$. From this it is seen that when $a = \frac{l}{2}$ the shear is zero, or that the shear at the centre

is nil, and the shear increases directly in proportion to the distance from the centre.

If in Fig. 8 we erect at the left abutment a vertical equal on any scale to $\frac{l - a}{l}$ W, and similarly at the right abutment a vertical equal to $\frac{a}{l}$ W, and draw horizontals through the extremities of these verticals until they meet a vertical erected at W, the ordinates thus obtained give the shear at any section to the same scale.

If in Fig. 9 we erect at each abutment a vertical on any scale equal to $\frac{w\,l}{2}$ and join the extremities of these verticals with the centre of the line representing the girder, the ordinate thus obtained at any section will represent the shear at that section to the same scale.

For a given intensity of load per unit of length, the greatest shearing force at any cross section of a girder takes place when the longer of the two parts into which that section divides the girder is loaded and the shorter unloaded.

"Let the ends of the girder Fig. 10 be called C and D, and the given cross section E; and let C E be the longer part, and E D the shorter part of the girder. In the first place let C E be loaded and E D unloaded. The load may be altered either by putting weight between D and E, or by removing weight between C and E. If any weight be put

between D and E, a force equal to *part* of that weight is added to the supporting force at D, and therefore to the shearing force at E; but a force

FIG. 10.

equal to the whole of that weight is taken away from that shearing force, and therefore the shearing force at E is diminished by the alteration of the load. If weight be removed from the load between C and E, the shearing force at E is diminished also, because of the diminution of the supporting force at D. Therefore any alteration from that distribution of the load in which the longer segment C E is loaded, and the shorter segment E D unloaded, diminishes the shearing force at E." *

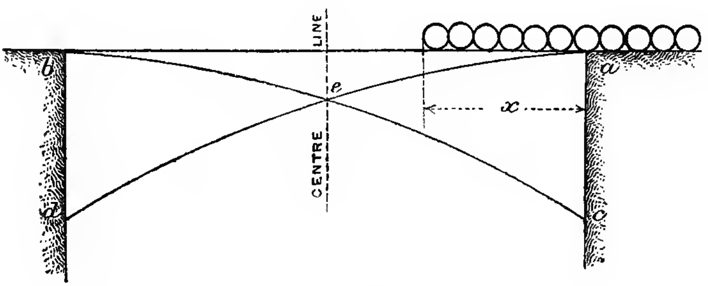
In Fig. 11, let a uniform load, whose length exceeds the span of the girder, pass over the girder, moving from right to left. Then if the head of the load is in any position distant x from the right abutment, the shearing force throughout the unloaded portion of the girder is equal to the reaction at the left abutment $= w\,x \times \frac{x}{2\,l} = \frac{w\,x^2}{2\,l}$.

As the load moves forward the shear at the head of the load increases as the ordinates of a

* Rankine, 'Applied Mechanics.'

parabola, and is given by the ordinates of the curve a, e, d, where $b\,d = \frac{w\,l}{2}$, which is the shear at the right abutment when the moving load completely covers the span. From what we have previously

FIG. 11.

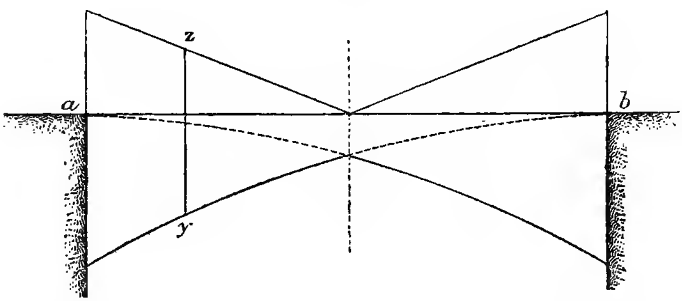


seen above, as soon as the head of the load passes the centre of the girder the shearing force, at whatever section the head of the load may be, is the maximum shearing force which this load could impose at that section; that is, from e to d, the ordinates of the curve at any section give the maximum shear which the moving load can impose at that section. If the moving load now goes from left to right, we have the corresponding curve $b\ e\ c$ giving the shears at the head of the load as it moves over, and in this case the ordinates to the right of the centre give the maximum shears which the moving load can impose at any section; therefore, whether the load moves from right to left or *vice versa*, the ordinates between the line $a\ b$ and the curve $c\ e\ d$, give the maximum shear at each section of the girder.

If we now consider the live and dead load together:

In Fig. 12, the ordinates above the line *a b* give the shear at each point for the dead load, and the ordinates between *a b* and the full curved line be-

FIG. 12.



low give the maximum shears at any point from the moving load; therefore the lines between the boundaries of the two shears such as *z y*, give the maximum shear at any point from both dead and live loads.

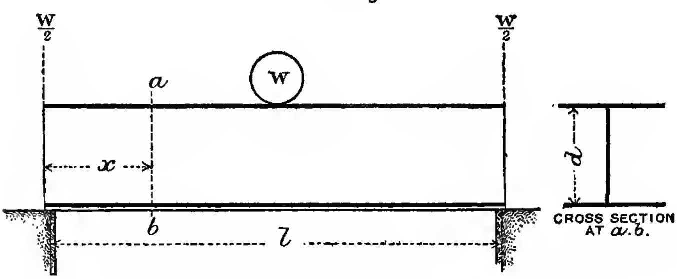
CHAPTER II.

STRAINS IN GIRDERS.

LET us now consider how a girder resists the effect of the bending moment at any point.

Let Fig. 13 represent a horizontal flanged girder with continuous web. Let l be the span, centre to

FIG. 13.



centre of bearings, let d be the depth between centres of flanges, and suppose a load W in the middle. Take a section, distant x from the left abutment. The forces keeping the portion of the girder between the left abutment and x in equilibrium are the reaction $\frac{W}{2}$, the horizontal forces of compression and tension in the top and bottom flanges respectively at a and b, and the shearing force at the section $a\ b$, neglecting, as is usual in English practice, any

horizontal strain that may be taken up by the plate-web. Take moments around point a and we have,

$$\frac{W}{2} \times x = \text{tension in bottom flange} \times d,$$

and taking moments around b we have

$$\frac{W}{2} \times x = \text{compression in top flange} \times d,$$

the shearing forces in each case disappearing. It will thus be seen that the compression in the top flange is equal to the tension in the bottom flange.

The left-hand side of both equations is the bending moment at the section $a\,b$; therefore we have,

The compression in top flange = tension in bottom flange = bending moment ÷ the depth of the girder.

Similarly, for any other section of the girder, the stress in either flange at any point is equal to the bending moment at that point divided by the depth of the girder; and in any horizontal flanged girder with continuous web, however loaded, the stress in either flange at any point is equal to the bending moment at that point divided by the depth of the girder.

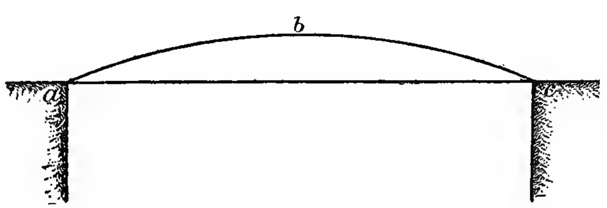
In a girder, therefore, loaded at the centre with a weight W, the stress in either flange at the centre is $\frac{W\,l}{4\,d}$.

In a girder loaded uniformly with a load w per unit of length, the stress in either flange at the centre is $= \frac{w\,l^2}{8\,d}$, or if the total load $w\,l = \mathrm{W}$, the stress in either flange $= \frac{\mathrm{W}\,l}{8\,d}$.

Since the stress in the flanges of a horizontal girder with a continuous web is always equal to the bending moment, divided by the depth of the girder, it follows that, if we have a curve whose ordinate at any point represents the bending moment at that point, the same curve, but to a scale d times greater than the scale used for bending moments, where d = depth of girder, will represent the stresses in the flanges.

Thus, in Fig. 14, if the ordinates of the curve $a\ b\ c$ represent the bending moments of a girder of

FIG. 14.



40 feet span and 4 feet deep, to a scale of 20 foot-tons to the inch, the ordinates of the same curve, but to a scale of 5 tons to the inch, give the stresses in the flanges.

CHAPTER III.

STRAINS IN SOLID BEAMS.

We shall now briefly consider the effect of transverse loads on solid beams.

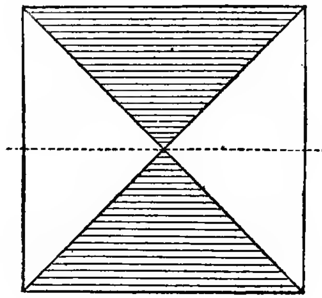
If a beam be supported at both ends and loaded transversely, the upper fibres are in compression and the lower in tension, and the neutral axis, i. e. the position where the fibres are neither in tension nor compression, passes through the centre of gravity of the section.

The determination of the strength of the beam depends on the theory that every fibre in a beam is strained in direct proportion to its distance from the neutral axis.

" Representing the effect of this condition geometrically for a square cross-section we are informed of the following facts. The neutral axis being at the middle of the depth, at that point the horizontal strain will be *nil;* increasing to the maximum amount at the extreme depth. If, therefore, we draw the diagonals of the square (Fig. 15) we obtain two triangles—shaded on the diagram—the width of which at any given distance from the neutral axis will be proportional to the horizontal strain on the

fibres at the same point, and the *area* of which will, consequently, be proportional to the sum of the horizontal action of all the fibres on each side of the

FIG. 15.



neutral axis. In short, the area of the triangle represents the equivalent area of metal, assuming the stress at all points to be equal in amount to that on the extreme fibres." *

The centre of gravity of each of these triangles is of course at a point two-thirds of the height of the triangle from the apex. We may, therefore, consider the whole of the area of each triangle concentrated at this point; and as in the case of a flanged girder, the stress in either flange was equal to the bending moment divided by the depth of the girder, in this case we may say the stress in each triangle is equal to the bending moment divided by the depth between the centres of gravity of the two triangles. In this case, therefore, the stress in each triangle will be

* B. Baker, 'Beams, Columns and Arches.'

$\frac{\text{B M}}{\frac{2}{3}d}$, and if we divide this by the area of the triangle (as in a flanged girder we divide the similar expression by the area of the flange) we get the stress per square inch to be $\text{B M} \div \left(\frac{2}{3}d \times \frac{d^2}{4}\right) = \text{B M} \div \frac{d^3}{6}$. The stress per square inch thus obtained is the stress per square inch in the extreme fibres. The expression $\frac{d^3}{6}$ is called the moment of resistance for this section; and for any section, the bending moment at that section divided by the moment of resistance for that section gives the stress per square inch in the extreme fibres.

As an example, suppose we have a bar of iron 6 inches square, on supports 10 feet apart, loaded with 5 tons in the middle; let us find the stress on the extreme fibres in the centre of the span.

The bending moment at the centre, due to the 5 tons in the middle, is $\frac{5 \text{ tons} \times 120 \text{ inches}}{4} = 150$ inch-tons. The weight of the bar will be, say half a ton; therefore the bending moment, at the centre, due to its own weight, is $\frac{0\cdot5 \times 120}{8} = 7\cdot5$ inch-tons; and therefore the total bending moment at the centre $= 157\cdot5$ inch-tons. The moment of resistance is $\frac{d^3}{6} = \frac{6^3}{6} = 36$, and therefore the bending moment divided by the moment of resistance $= \frac{157\cdot5}{36}$

= 4·4 tons, which is the stress per square inch in the extreme fibres.

In beams of non-symmetrical cross-section we shall be able to find the moment of resistance with very little more trouble than in symmetrical sections, if we remember the two fundamental rules:— (1) The neutral axis passes through the centre of gravity of the section. (2) Each fibre is strained in direct proportion to its distance from the neutral axis.

As an example of finding the strength of an unsymmetrical section, we shall take the case of a trough floor which has been used on a large number of bridges, and of which the section of one flute is shown full size in Plate I. The pitch of the troughs is 2 feet 4 inches, centre to centre, and they are connected at the top table by 8-inch by $\frac{5}{8}$-inch plate. There are no rivets in the bottom table, and as the top will be in compression, assuming that the troughs are not fixed at the ends, the rivet holes there need not be considered.

To save work we shall consider only half of one flute, that is, the portion shown between the thick dotted lines. The strength of the whole flute will of course be double the strength of this portion.

There is no difficulty in finding the position of the centre of gravity of this section. The area of half the trough without the cover on top table is 11·953 square inches, and the area of the cover 2·5

square inches. Taking the moment of each of these around the bottom edge we have :—

Moment of trough	=	11·953 sq. inches	× $4\frac{1}{16}$	inches	= 48·559
Moment of cover	=	2·5 ,,	× $8\frac{7}{16}$	,,	= 21·094
				Total Moment	= 69·653

If we then divide the total moment by the total area we get the height of the centre of gravity of the section above the bottom edge, thus :—

$$\frac{69\cdot653}{14\cdot453} = 4\cdot82 \text{ inches.}$$

Now, remembering that the stress in each fibre is proportional to its distance from the neutral axis we see that the maximum stress must be at the bottom fibre. For the portion below the neutral axis, consider the horizontal fibre *a b*; draw vertical lines from its extremities to the bottom edge or to the line of the bottom edge produced, join the two points on the bottom edge thus obtained to the point *o*, which is the centre of the beam at the neutral axis; these two lines intercept a certain length on the horizontal fibre *a b*, or the horizontal fibre *a b* produced. Making a similar construction for all the fibres below the neutral axis we obtain the shaded area in diagram which represents the equivalent area of metal, assuming the stress at all points to be equal to that on the extreme fibre in a similar way to that used for the square section. For the portion above the neutral axis the construction is

similar, except that, as the extreme fibre is closer to the neutral axis than the extreme fibre in the lower portion; we must draw our vertical lines to a horizontal line which is the same distance from the neutral axis as the extreme fibre in the lower portion of the section is. The construction is shown for the fibre *c d*. The shaded portions in the figure were obtained by taking fibres one-eighth of an inch apart horizontally, and is quite close enough for sections of this kind. It may also be noted that any other point on the neutral axis might be used to radiate to instead of the point O.

It is evident now that if the construction is correct, the shaded figures on each side of the neutral axis must be equal. On taking out their areas with a planimeter they are each found to be practically 4·55 square inches.

We have now to find the centre of gravity of each of these shaded figures, and this is done quickly by cutting them out in cardboard, and pinning each one to a vertical board or wall. If a line of thread with a weight on it is now hung from the same pin, this line will pass through the centre of gravity of the figure if the cardboard is free to rotate on the pin. If the cardboard is hung afterwards from the pin by another hole in the cardboard the plumb line will again pass through the centre of gravity, and the intersection of the lines thus traced on the cardboard by the plumb line will give the centre of gravity of the section.

In this way, in the sections under consideration, we have found that the centre of gravity of the upper shaded figure is 3·13 inches above, and of the lower figure 4·02 inches below the neutral axis. The moment of resistance of the section is the area of either figure multiplied by the distance between the centres of gravity of the two figures. We have found that the area is 4·55 square inches and the distance between the centres of gravity is 7·15 inches, therefore the moment of resistance of the section is

$$4·55 \times 7·15 = 32·53$$

and therefore the moment of resistance for the whole flute will be 65·06 in inch units.

We can now find the *theoretical* strength of any solid beam, and if we know the direct ultimate strength of the metal, we can calculate what load will produce a stress on the outside fibre equal to its ultimate strength; theoretically the beam ought then to break, but practically it will not do so. The increase of strength is due to the lateral adhesion of the fibres, and the proportionate increase of strength is different for different cross sections, and for different metals. In rectangular and round cross sections the strength may be increased above that found by theory from 60 to 70 per cent. in wrought iron or steel, and to 125 per cent. in cast iron. As the form of section approaches the **I** section the increase of strength gets less and less, and in the

calculations which will have to be made in these pages for bridge floors and floor beams will be neglected.

This increase of strength above that shown in theory is however of vital importance in beams approximating to rectangular and round sections, and in all sections where a great proportion of the metal is near the neutral axis.

CHAPTER IV.

LOADS ON BRIDGES.

WE have now gone through nearly all the theory which is necessary for the construction of such bridges as we propose to deal with; any other points which may appear to require explanation will be dealt with as arrived at.

LOADS ON BRIDGES.

The *Dead Load* on the main girders of a railway bridge may be divided into four divisions:—1st, the weight of the main girders themselves; 2nd, the weight of iron or steel floor, including cross-girders and longitudinals, if used; 3rd, the weight of the permanent way, made up of rails, chairs, sleepers and fastenings; 4th, weight of ballast, concrete, &c., if used.

In the spans which are now under consideration the weight of the main girders is a very small portion of the total load which the girders have to be designed to carry. As the span increases the weight of the main girders becomes a larger proportion of the total weight, but in a 50-foot span for a double line, the weight of the main girders would

not be more than about 8 per cent. of the total weight, and in smaller spans of course a still less percentage.

The following table will give a sufficiently close approximation, for purposes of calculation, to the weights of main girders of different spans for a double line, when the track is carried on a floor resting on the flanges of girders. As the weight of main girders, as above shown, is a small percentage of the total load, this table may be used for either iron or steel, without great error, although iron would be a little more, and steel a little less.

BRIDGES WITH TWO MAIN GIRDERS.

Span in Feet.	Weight in Cwts. per foot run.	Span in Feet.	Weight in Cwts. per foot run.
20	3·5	50	7·0
25	4·0	55	7·7
30	4·5	60	8·5
35	5·0	70	9·6
40	5·6	80	10·8
45	6·4		

BRIDGES WITH THREE MAIN GIRDERS.

Span in Feet.	Weight in Cwts. per foot run.	Span in Feet.	Weight in Cwts per foot run.
20	4·0	45	7·5
25	4·7	50	8·1
30	5·5	55	8·7
35	6·3	60	9·3
40	6·9		

The weight of steel or iron floor, now that so many different kinds of floors are in use, varies very much, and the weight of the actual floor used for any bridge should be worked out before designing

the main girders. For a double line with three main girders and light troughs it may be as low as five cwts. per foot run, and with two main girders and heavy troughs it may go up to nearly double this amount.

The weight of permanent way is of course a weight easily obtainable for any particular railway, varying only a little as cross-sleeper or longitudinal sleeper is used on the bridge.

The weight of ballast and concrete, if any, varies of course with the materials used for these purposes in the neighbourhood. In the case of ballast, it will be well to allow for ballast when wet, which is very much heavier than when dry.

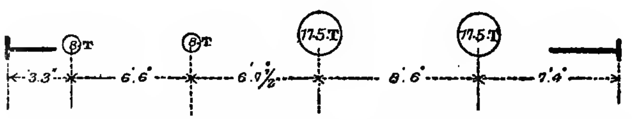
The *Rolling Load* in small-span bridges is by far the most important element of the total load. The first, thing, therefore, in designing bridges for any particular railway is to get all particulars of the heaviest engines which are used on this particular line, or which may be liable to go over it. The engine having the greatest total weight may not be that which will impose the greatest stresses on very small spans, as an engine whose total weight is less may possibly have a heavier load on its driving wheels. However, we can generally decide on one engine which we can use for all spans.

The easiest way now to deal with the weight of the engine which we have decided on is to construct a table giving the equivalent uniform loads per foot run which would produce the same effect for different

spans as the maximum effect produced by this engine. We can work this table out for spans, increasing by, say 3 feet or 5 feet, and then we can interpolate the values for the intermediate spans.

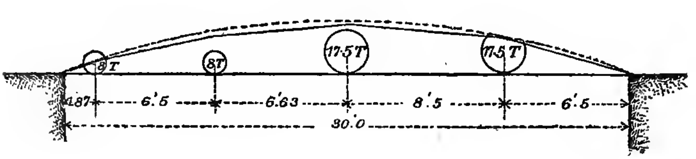
Let us take the heavy tank engine, as shown below in Fig. 16, where the weights on the axles are respectively 8 tons, 8 tons, 17·5 tons and 17·5 tons, and the distances apart of the axles as shown.

FIG. 16.



If this engine is placed on a 30-foot span, the position which gives the maximum bending moment is as in Fig. 17.

FIG. 17.



The diagram of bending moments is shown by full lines. If we now draw a circle, shown dotted in Fig. 17, passing through the points of support and just clearing the diagram of bending moments, this circle will be the curve of bending moments for a uniform load which will have practically the same effect on the girder as the engine load we are considering. By this method, the following table gives

the equivalent uniform load per foot run for each track which would produce same effect as the tank engine shown above, two such engines being coupled together, for the spans above 35 feet.

Span in Feet.	Equivalent Uniform Load per foot run in Cwts.	Span in Feet.	Equivalent Uniform Load per foot run in Cwts.
20	47·0	50	33·2
25	43·8	55	32·9
30	41·0	60	32·5
35	38·2	70 . ..	32·1
40	36·1	80	31·9
45	34·3		

The equivalent uniform loads per foot run, given in the above table, refer to calculations as regards stresses in both flanges, and webs.

The distribution of the actual loads which would give the maximum shear at any section, would of course be quite different from the distribution which gives the maximum bending moment, as explained in pages 9 and 12.

Suppose we have a single concentrated load W in the centre of a span. The total uniform distributed load which would give the same maximum bending moment would be 2 W. For the single concentrated load W in the centre, the shear close to either abutment is $\frac{W}{2}$; for the uniform distributed load, the shear close to either abutment would be W. Therefore, the uniform distributed load, which gives the same maximum bending moment as the concentrated load in the centre, gives double the shear at either

abutment. If the concentrated load, however, is now rolled across the span, the shear when it is close to either abutment is W. We see, therefore, that in this case the equivalent uniform distributed load does give the maximum shear as well as the maximum bending moment. Although, when two or more rolling loads pass over a span, a similar statement will not hold absolutely, still without any great error the table above given of equivalent uniform distributed load, may be used for calculating the maximum bending moments and the maximum shears at any section; in the former case the uniform distributed load covering the whole span, and in the latter case the uniform distributed load passing from one side of the span to the other.

The effect of wind pressure on small bridges such as we are considering is not important, and the increase of stress in flanges due to it is so small, that in bridges with a continuous floor it need not be considered. The horizontal force of the wind is resisted by the bridge acting as a horizontal girder of a depth equal to the width of the bridge, the main girders acting in part as flanges, and the continuous floor as web. It is obvious that in small spans the depth of this horizontal girder is very great in proportion to the span. If there is not a continuous floor horizontal bracing must be introduced. When there are cross-girders firmly attached to the flanges of the main girders, flat bars crossing each other, riveted to the flanges of the main girders at the

ends of the cross girders, are sufficient for this purpose. With a continuous floor it is necessary that the plating should be *really* continuous, so as to be able to transmit the stresses due to the wind, and that it should be well attached to the flanges of the main girders. As before stated, the pressure due to wind in small spans is very little, although a most important element in bridges of large spans.

CHAPTER V.

WORKING STRESSES IN STEEL AND IRON.

WE have now to consider the safe working stresses in steel and iron. This is one of the most important things to be considered in bridges of this class, as on the stress which is allowed depends the strength of the bridge altogether, and the fact that the great majority of railway bridges *are* small-span plate girders makes it all the more important. The Board of Trade requirements are that the maximum stress per square inch caused by the combined rolling and dead load shall not exceed $6\frac{1}{2}$ tons in steel and 5 tons in iron. This rule pays no attention to the ratio of the dead load to the live load, which is most important in bridges of the class we are considering.

If the Board of Trade rule for steel bridges was uniformly adopted for all spans, we might have in the case of the flanges of the main girders of a bridge of 20 feet span, the stress due to the dead load one ton per square inch, and that due to the live load $5\frac{1}{2}$ tons per square inch. The *range* of stress would, therefore, be $5\frac{1}{2}$ tons, assuming that the stress due to the live load was exactly that due to the weights

shown to be on the axles of the locomotive when on the weighbridge. In the case of the flanges of the main girders of a bridge of 200 feet span, we might have the stress due to the dead load $3\frac{1}{4}$ tons per square inch, and that due to the live load a similar amount. The *range* of stress in this case would, therefore, be $3\frac{1}{4}$ tons per square inch. Now from experiments made by many engineers, it has been found out that the range of stress is one of the principal points which determine the life of steel and iron structures, so that if the range of stress is small a large unit strain may be adopted. From this reason alone, therefore, if $6\frac{1}{2}$ tons per square inch on steel would be a proper stress to adopt for the flanges of the main girders for a 200-foot span, it would evidently be an excessive stress for those of a girder of 20 feet span. In addition to this, in small-span bridges, the lurching and jumping of a locomotive due to the permanent way not being in good condition, or from other causes, may seriously increase the stresses in the main girders, while in large spans the increase in stresses in *main* girders due to this cause may be almost neglected, although of course, provision must be made for it in whatever type of floor may be adopted for the bridge.

For *steel* structures we may, therefore, take as the allowable tensile stress per square inch on net section for main girders, $4\frac{1}{2}$ tons for 20-foot spans and under, 5 tons for 30-foot spans, and $5\frac{1}{2}$ tons for 80-foot spans, the stresses for intermediate spans being

proportional, and no tensile stress on cross-girders or rail-bearers exceeding $4\frac{1}{2}$ tons per square inch. The corresponding compressive strains may be $3\frac{3}{4}$ tons, $4\frac{1}{4}$ tons and $4\frac{3}{4}$ tons per square inch on gross section, provided that in the case of compression flanges there is either ample width of flange or that they are braced to resist buckling, and that in thin wide flanges they are properly stiffened at the edges by angle bars or in other ways.

The shearing stresses on rivets should be about two-thirds, and the bearing stresses about one and a half times the corresponding tensile stresses.

For *iron* structures we may take the working stresses at 20 per cent. below those given for steel.

CHAPTER VI.

MATERIALS.

THOUGH a few bridges are still made of wrought iron, the majority of those now erected are made of steel. We shall therefore make a few remarks on this metal first.

The following are the conditions which good bridge material ought to satisfy.

STEEL.

The steel must be made by the *open-hearth* process. Strips cut lengthwise or crosswise shall have an ultimate tensile strength of not less than 28 tons, and not more than 32 tons per square inch of section, with an elongation of 20 per cent. in 8 inches ; and when heated uniformly to a low cherry-red and cooled in water of 82° F., strips $1\frac{1}{2}$ inches wide must stand bending double to a curve, of which the inner radius is one and a half times the thickness of the plate. No work ought to be done on any of the material after it has cooled down to the condition known as "blue heat," which is about from 400° F. to 600° F., and any steel which has been smithed or bent should be subsequently annealed. Rivet steel

is a softer steel, and ought to stand a tensile stress of not less than 26 tons and not more than 28 tons, with an elongation of 25 per cent. in 8 inches.

Wrought Iron.

Wrought iron must have a minimum ultimate strength of 21 tons per square inch of section, and an elongation of 8 per cent. in 8 inches with the grain, and a minimum ultimate strength of 18 tons and 3 per cent. elongation across the grain. Rivet iron ought to stand a tensile stress of 23 tons per square inch, with an elongation of 20 per cent in 8 inches.

All ironwork to be rolled perfectly smooth without blisters or other imperfections.

General Conditions of Steel and Iron.

The ends and edges of all plates (except where rolled edge plates are used) shall be planed, and must butt perfectly true, and angles and tees must be either sawn at the ends, or be neatly finished off after being smithed. All rivet-holes must be drilled, and the rivet-holes in adjacent plates and bars must come opposite each other without distressing the plates and bars with drifting. The work ought to be erected and put together in the manufacturer's yard, and all the parts properly fitted together before removal to site of erection. Each main girder for bridges up to 40 feet span, or even more, is often riveted completely together in the yard, and then taken in one

piece to the site; this saves something on riveting, as of course it can be done cheaper in the yard than by sending out men to do it at the site. When the girders are erected in the yard they ought to be put together with their proper camber.

In designing small bridges, all smith's work should be reduced to a minimum, and no welding should be allowed. Small angles and tees with joggled ends, such as for stiffness in small girders, can generally be avoided by packings. Rivets should have large cup heads, and should be concentric with the holes, and all the work should be well bolted up with long spanners in front of the riveting. It will not do to bolt up a certain length, and then rivet that length; the bolts ought always to be tightened up just in front of the riveting, as the process of riveting in drawing the plates together, particularly in hydraulic riveting, loosens the bolts in front of it. Bad bolting up is one of the most fruitful causes of loose rivets.

In designing bridges, have as few different sections as possible. It may seem unnecessary to say that care must be taken in the drawings, not to show rivets where there is not sufficient room to put them in, as well as to snap them. All iron and steel should be tested at the works where rolled.

After the work is erected in the manufacturer's yard all the pieces should be carefully marked so as to facilitate erection at the site.

After temporary erection, the bridge, after being

well scraped to get rid of all rust, should receive two coats of best lead or oxide paint, and when permanently erected two more coats. Some parts of the bridge from which water may possibly be some time in getting away are often covered with pitch and tar instead of paint.

CHAPTER VII.

KINDS OF BRIDGES.

In plate-girder bridges for a double line up to 40 feet, and sometimes even for larger spans, the usual type of bridge now adopted is that of three main girders with trough floors. This is more particularly the case when the headway is limited, as then with three girders a comparatively shallow transverse trough resting on the bottom booms of the main girders is sufficient to take the traffic of each line of rails. When more head-room is available, an excellent bridge is formed with three main girders and cross girders with light longitudinal troughs. It is evident that for spans up to 40 or 50 feet a saving will always be effected by using three main girders in preference to two, as although the total weight of the main girders is increased, there is more than a corresponding diminution in the weight of the floor. Beyond these spans it will generally be more advantageous to use only two main girders with either transverse trough flooring, or with cross girders and longitudinal troughs, or with cross girders and rail-bearers and plate floor. It must be remembered that in using three main girders the width of the bridge will have to be in-

creased, if the main girders extend more than 2 feet 6 inches above rail level. The Board of Trade rule is, that "No standing work (other than a passenger platform) to be nearer to the side of the widest carriage in use on the line than 2 feet 4 inches, at any point between the level of 2 feet 6 inches above the rails, and the level of the upper parts of the highest carriage doors. This applies to all arches, abutments, piers, supports, girders, tunnels, bridges, roofs, walls, posts, tanks, signals, fences and other works, and to all projections at the side of a railway constructed to any gauge."

This rule also fixes the distance apart of parapets on each side of bridge. If we take the width of the widest carriage as 8 feet 9 inches, and $2\frac{3}{4}$ inches as the width of the head of rail, we get the minimum distance between parapets as below:—

	ft.	in.
From parapet to side of carriage	2	4
„ side of carriage to centre of track ..	4	$4\frac{1}{2}$
„ centre to centre of tracks	11	2
„ centre of track to side of carriage ..	4	$4\frac{1}{2}$
„ side of carriage to parapet	2	4
Minimum distance apart of parapets ..	24	7

Very many bridges, however, are made with a clear distance of 25 or 26 feet between parapets. If the centre girder, however, is higher than 2 feet 6 inches above rails, these figures are much increased. In the country, parapets are generally made 4 feet 6 inches above rails, and either open or close, but in towns they are generally made close, and 6 feet

above rails. In long viaducts recesses for plate-layers have of course to be provided at intervals.

In spans of 10 to 15 feet, longitudinal troughs laid from abutment to abutment are often used with a light face girder on each side to finish up the troughs, and to carry a light hand-rail. Bridges of these spans are also often made with a box trough for each rail, and a curved plate floor attached to the troughs and face girders.

Before going further, we should like to make some remarks about the trough floors now usually adopted, and of which different kinds are shown in the plates at end of book. These may be broadly divided into transverse troughs and longitudinal troughs. We shall first consider the former kind of trough. These are either stamped, or built up from different sections. Those generally adopted where *three* main girders are used, are stamped by hydraulic power, and those used between *two* main girders are generally built up, as owing to the greater depth required, stamping is more difficult.

With these transverse troughs a great diversity of opinion exists as to whether an ordinary ballasted road with cross sleepers should be laid, or whether the rails should be laid without ballast on longitudinal timbers resting on the top of the troughs.

A floor laid *without ballast* has the following advantages:—

1. There is less dead weight to carry.

2. The load on the axles of the locomotive is a

little better distributed over the troughs than in a ballasted road.

3. Rain-water runs away quicker.

4. The condition of the floor can be seen easily, and it can be painted or tarred as required.

The principal advantages of a ballasted floor are:—(1) There is no break in the ordinary permanent way, which is a very great advantage. (2) It is not quite so dangerous as a non-ballasted floor in the case of a locomotive leaving the track.

The stresses on these floors ought to be kept low, as the range of stress is considerable. A stress of 4 tons per square inch is as much as ought to be allowed on these floors when constructed in steel, and $3\frac{1}{2}$ tons in iron.

In transverse troughs, the question of the amount of load which each trough has to carry is a very important one. For instance, suppose that there is 17·5 tons on an axle which is over the centre of a trough, how much of that load comes on that trough, and how much is distributed on to adjacent troughs? It is evident that if the axle load was placed on the trough, without the intervention of rail or sleeper, that the one trough would have to carry almost all the load, as the connection between troughs is not stiff enough to allow adjacent troughs to help much. When we have a rail between the load and the trough the question is, however, quite changed because the stiffness of the rail and of longitudinal sleeper (if used) comes in. The amount of load which the

trough will then have to carry varies inversely as the stiffness of rail and longitudinal sleeper, and directly as the stiffness of the trough itself. The rail and sleeper, therefore, distribute the load over adjacent troughs. The amount of this distribution will vary with each kind of rail and trough, and depends a good deal on the state of the permanent way.

Not very much has been written on the subject of this distribution, probably because it is rather indefinite, but many engineers assume that with cross sleepers the load is distributed over three sleepers, and it can be proved, that with a locomotive on a transverse trough floor with longitudinal sleepers and troughs of 2 feet pitch, that not more than about three-tenths of the load on one axle can come on any trough even in the worse case, and in the case of a transverse ballasted trough floor, with troughs of 1 foot 8 inches pitch, not more than about seven-twentieths of the weight on one axle can come on a single trough. We have, therefore, assumed for the bridges which are to be afterwards considered that half the load on one axle is the maximum that need ever be provided for in one trough, when the pitch of the trough is not less than 2 feet or more than 2 feet 4 inches, and this assumption will probably in most cases be above what actually takes place.

We ought, however, though making the assumption, not to lose sight of the fact, as stated already, that the distribution varies with each kind of rail and

trough used, and is evidently better with longitudinal than with cross sleepers.

It might be well to draw attention here to the necessity of keeping the permanent way in the best possible condition over bridges, and particularly to the necessity of the rail joints being well looked after.

When a ballasted floor with transverse troughs is adopted, the ballast is usually spread so that there shall be a minimum of 3 inches of ballast between the top of the trough and the bottom of the sleeper. Some bridges, when the headroom is extremely limited, have the sleepers laid in the troughs with about 3 inches of ballast between the bottom of the trough and the bottom of the sleeper. By this means about 8 or 9 inches can be saved in headway, but unless absolutely necessary this method ought not to be adopted, as if there is any defect in the drainage of the troughs the sleepers rot quickly, and they are also more difficult to renew, as they cannot be drawn out sideways, so that a rail has to be lifted to get the sleeper in or out.

One of the greatest difficulties with transverse trough floors is to make them watertight at their junction with the main girders. Their drainage is generally provided for by a hole through the bottom of the trough near the central girder, from which the water is taken in a small water channel fixed underneath the troughs for the whole length of the bridge to a down-pipe at the abutment. The central girder is generally bedded an inch lower than the

side ones to allow the water to drain to the holes in the troughs. Some transverse floors, particularly Hobson's flooring (Plate III. Fig. *7a*), are sometimes levelled up with concrete on which is laid an inch or an inch and a half of asphalt over the whole floor with a slope toward each abutment where the water is collected. If the bridge is on a gradient, the slope due to that gradient is generally sufficient for the purpose, and the water is then collected at one abutment.

Longitudinal troughs with cross girders are possible when headroom is not very limited. As the troughs are generally much shallower than transverse troughs, the weight of ballast on the troughs is not very great. This makes a very good type of floor, and the drainage from the troughs can easily be run to each abutment by keeping the troughs a little high in centre of bridge. A good floor and one which until the introduction of trough floors was almost universally used, is formed by cross girders, rail-bearers and a plate deck with sufficient ballast to take the ordinary permanent way, but the drainage sometimes presents a little difficulty. This kind of floor can also be used without ballast by laying the rails on longitudinal sleepers on top of the rail-bearers.

We have already referred to four main girders in the shape of built-up troughs, one under each rail, being used for very small spans; for larger spans, when there is sufficient headroom it will often

be found to be economical and convenient to use four girders, one under each rail, of ordinary **I** section, with a plate floor attached to the top flanges and to a light face girder.

Two main girders, one under the outside rail of each track, with cross girders and rail-bearers to carry inner rails, is also an economical bridge when there is sufficient headroom.

The drainage of all these different types of bridges is very important.

In towns, a light sheet-iron floor is often hung under the bridge, as an additional precaution, to take away drippings from the bridge floor to the sides.

As a summary of the few preceding pages:— When headroom is very limited three main girders, with transverse troughs, or four main box troughs must be used; in the majority of cases the former class of bridge is preferable. When a little more headroom is obtainable, three main girders with cross girders, and a floor consisting of either rail-bearers and curved plates, or longitudinal troughs, or two main girders with transverse troughs, can be used; in this case the three main girders seem preferable certainly for spans up to 40 feet. When there is no restriction as to headroom a variety of bridges can be obtained, and the most economical up to spans of 30 or 40 feet will probably be that with one main girder under each rail. It will be found in many cases that local conditions are the principal reasons for fixing the type of bridge to adopt.

CHAPTER VIII.

DEPTH OF GIRDERS.

ONE of the most important and one of the first points in designing a girder is to settle what its depth is to be. We know that by increasing the depth of a girder we diminish the stress on the flanges, and consequently their section may be reduced; but at the same time, we shall probably increase the quantity of material in the web, and the weight of stiffeners will also be increased. If the cross section of a girder at all points could be made exactly proportional to the stress which it has to stand, it would be an easy matter to find out the economical depth. This, of course, is impossible in the flanges, as we have to deal with plates of uniform rectangular section and of certain maximum lengths, and we cannot diminish their section to zero at the abutments. As regards the webs; in these, plates less than $\frac{3}{8}$-inch in thickness are seldom used, so that we have here a minimum thickness, and we have also the extra weight due to the stiffeners. It will, therefore, be easily seen that the determination for the girders of any bridge of the ratio of depth to span so as to have the minimum amount of material

in the girder to carry a given load per foot run is not a problem of which a theoretical solution can be obtained. With heavy loads per foot run it is more economical to make girders deeper than would be made with light loads per foot run. Local conditions often fix the depth of girders. Consider the case of a bridge of three main girders with a trough floor ; if the flanges project more than 2 feet 6 inches above rail level, the width of the bridge must be materially increased beyond what is necessary when they are kept below this level. It will sometimes be found, therefore, that it is more economical to make the girders shallow rather than increase their distance apart. Again, as with three main girders, the weight on the centre girder is almost double that on each of the side girders ; the economic depths ought to be different, but for reasons of construction they are generally kept the same. In the case of three main girders, up to 40 feet span, we find that a depth of one-tenth to one-eleventh of the span, and with two main girders up to 80 feet span, a depth of about one-eighth to one-tenth of the span will be the most economical depths, and in the case of the former, even with these comparatively shallow depths, it will often be found difficult to reduce the section of the side girders as low as required.

The breadth of the flanges is generally governed by the width which is considered necessary to prevent the compression flanges from buckling laterally, both flanges being generally made the same width.

In spans from 20 feet to 60 feet, the widths may vary from 12 inches to 21 inches, and for spans over 30 feet angle bars along the outside of the compres sion flanges, in addition to those connecting the web to the flange, will add to the stiffness. If the compression flanges are stiffened in other ways, such as by a stiff plate floor on the top flanges, the widths of flanges can be made much smaller if necessary.

Girders are nearly always erected with a camber. This is done more for appearance sake than any other reason. The amount of camber is generally made sufficient to balance any deflection caused by the heaviest loads which may come on the girder, and any sagging caused by imperfections in workmanship. The camber generally given to railway plate girders varies from $\frac{1}{2}$ inch for 20-foot spans, to about 2 inches for 80-foot spans.

CHAPTER IX.

ORDINARY SIZES OF PLATES AND ANGLE BARS IN STEEL AND IRON.

PLATES and bars of ordinary market sections and lengths should be used as far as possible, though of course sometimes a saving may be obtained by paying extra for special sizes to do away with covers, or for other reasons.

It is well to remember that the difficulty of handling very thin plates of large area is often a reason for using such plates in smaller sizes than would be used with thicker plates.

STEEL.

The following table gives the maximum dimensions up to which steel plates can be obtained from one of the best Scotch makers without extras.

	$\frac{1}{4}$ in. and under $\frac{5}{16}$ in.	$\frac{5}{16}$ in. and under $\frac{3}{8}$ in.	$\frac{3}{8}$ in. and under $\frac{7}{16}$ in.	$\frac{7}{16}$ in. and under $\frac{1}{2}$ in.	$\frac{1}{2}$ in. and under $\frac{9}{16}$ in.
	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.
Length ..	20 0	23 0	26 0	29 0	32 0
Width ..	4 6	5 0	5 6	5 10	6 2
Weight ..	7 cwt.	10 cwt.	13 cwt.	17 cwt.	21 cwt.

	$\frac{9}{16}$ in. and under $\frac{5}{8}$ in.	$\frac{5}{8}$ in. and under $\frac{3}{4}$ in.	$\frac{3}{4}$ in. and under $\frac{7}{8}$ in.	$\frac{7}{8}$ in. and under 1 in.	1 in. and under $1\frac{1}{4}$ in.
	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.
Length ..	32 0	35 0	35 0	35 0	35 0
Width ..	6 6	7 0	7 0	7 0	7 0
Weight ..	26 cwt.	32 cwt.	40 cwt.	45 cwt.	50 cwt.

From this table, for plates of a certain thickness, if the width of a plate is fixed we can immediately find out from the maximum weight what the maximum length of plate, and if the length is fixed we can find the maximum width, which can be obtained without extras. By payment of extra charges these lengths and widths can be increased, the extra for length being 5*s.* per ton for every 5 feet, or part of 5 feet, over lengths given in table, and for width 5*s.* per ton for every 3 inches or part over widths given in table. There is generally an extra charge for plates under 12 inches wide, and some makers make an extra charge for plates under $\frac{3}{8}$ inch thick. By special arrangements plates of almost any required size can be obtained.

Plates with rolled edges, which obviate the necessity of planing the edges, are rolled by one or two makers up to about 30 inches wide. The extra charged for these plates is about 5*s.* a ton, when the plates are not below $\frac{3}{8}$ inch, or above $\frac{3}{4}$ inch in thickness.

Angle bars under 11 united inches, and over 6 united inches, and in lengths up to about 50 or 60

feet, can be obtained, without extra, of all ordinary thicknesses; above 11 united inches, the extra is 5*s.* per ton per inch, and below 6 united inches, 10*s.* per ton per half inch. T bars can be obtained without extras between 6 and 10 united inches, and for about the same lengths as angle bars.

It will be well to remember that these extras are liable to slight alterations from time to time, and in different districts; and there are also some slight differences in the extras charged by different makers. For very large orders a certain quantity of plates and bars of extra dimensions can generally be obtained at ordinary rates. The figures above given all refer to the Scotch steel-makers, and the sizes and weights of plates and angles rolled free of extras by the North of England makers are often not so great.

Iron.

No plate should exceed 10 cwt. in weight, or 20 feet in length, and the width should not exceed 4 feet 6 inches. This applies to all plates from ¼ inch up to 1 inch in thickness. Extras are charged of about 10*s.* per ton for every cwt. over 10 cwt.; 2*s.* 6*d.* per ton for every foot over 20 feet long, and 5*s.* per ton for every inch over 4 feet 6 inches wide.

Angle and T bars can be obtained from 4 united inches to 9 united inches without extra, and from 30 to 40 feet long.

The same remarks as to alterations in extras and dimensions will apply to iron as well as to steel, but the differences in iron are more marked among different makers. Some North of England firms roll without extras a good deal more liberally than the above figures.

CHAPTER X.

JOINTS IN PLATES, ANGLES, ETC.

Butt joints are almost the only joints which have to be dealt with in girder work, and these are made either with a single cover, as in Fig. 18, or with double covers as in Fig. 19. In the former case the rivets are said to be in single shear, as only one section of each rivet would have to shear to cause the

FIG. 18.

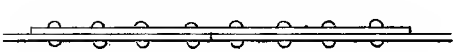
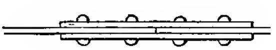


FIG. 19.



joint to fail by shearing of rivets, while in the second case each rivet would have to shear at two sections to cause failure, and the rivets in this latter case are said to be in double shear. For single covers the cover is generally the thickness of the plates to be united, and for double covers each cover is half the thickness of the plates, but both covers are often made thicker than this, especially with thin plates.

As the allowable shearing stress on rivets is only two-thirds of the allowable tensile stress on plates, angles, &c., the total shearing section of rivets in a tension joint ought to be one and a half times the

net section of one of the plates to be united. The allowable shearing stress on rivets is rather less than the allowable compressive stress on plates, angles, &c.; but in work where the ends of the plates are planed and butt, which ought to be the case in all good girder work, the total shearing area of rivets in a compression joint is generally made equal to the gross section of one of the plates to be united.

The bearing area of a rivet, which is the diameter of the rivet multiplied by the thickness of one of the plates to be united, must also be considered, and the stress must not exceed the working stress given previously.

For connecting plates less than half an inch thick, rivets of three-quarters of an inch in diameter, and for half-inch plates, and those under three-quarters of an inch, rivets of seven-eighths of an inch in diameter are generally used.

When rivets of large diameter are used for connecting thin plates, it is difficult to get sufficient bearing area without putting in so many rivets that shearing stress per square inch is very small.

The distance between the edges of adjacent rivet-holes, or between the edge of a rivet-hole and the edge of a plate, should not be less than the diameter of the rivet for drilled work, and if the work is punched these distances ought to be increased. As a general rule in girder work, except with very small rivets, it will not be found necessary to pitch rivets closer to each other than two and a half inches,

centre to centre, and that only in exceptional cases. Four-inch centres is the usual pitch for all the straight work in girders, and this will generally be found sufficient, but of course the correct pitch ought always to be worked out, to be sure that it is not less than this amount.

CHAPTER XI.

THREE MAIN GIRDER BRIDGE.

We shall now proceed to work out a few examples of bridges, such as will be met with in everyday work. They shall all be considered as made of steel.

The first we shall take will be a three-girder bridge, for a double line of rails of clear span of 30 feet, the details of which are shown in Plate II., and a small general plan of which is shown in Fig. 5 of the same Plate.

This bridge is on a slight skew, and we shall give the girders a bearing of 2 feet 6½ inches along the centre line of the girder. The distance from centre to centre of bearings, which will be the effective length of the girder for purposes of calculation, may be taken as 32 feet 6 inches.

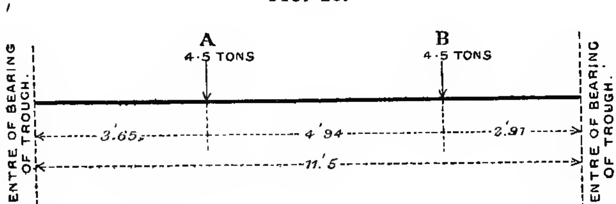
Both outside girders are of course comparatively lightly loaded, so that it will be economical to make the girders rather shallow, and we have therefore made the effective depth of all the girders three feet.

The floor will be as shown in Plate II., viz. :— transverse troughs with pitch 2 feet 4 inches, total

depth 8 inches, and thickness of metal ½ inch. The floor is unballasted, and longitudinal sleepers are used. The parapets will be as shown in Fig. 6 of Plate, the distance from outside to outside of side girders being 25 feet. The distance of side girders from centre girder, centre to centre, will therefore be 11 feet 10½ inches.

We shall first consider the strength of the floor, and for purposes of calculation will consider the troughs as free at the ends. It will be noticed that as there is the ordinary six-foot way between the two tracks, the lines of rails do not come centrally between the outside and inside girders. Taking the weight on an axle as 17·5 tons, and assuming as before explained that one trough takes half the weight on an axle, we have 4·375 tons on each trough under each rail as rolling load; if to this we add 0·125 tons, though rather too much, as the additional

FIG. 20.



weight at each point due to rails, chairs, sleepers and fastenings, we have the weights and their positions on a trough as given in diagram above. The effective span of trough from centre to centre of bearings may be taken as 11 feet 6 inches.

The reaction at left bearing

$$= \frac{\overset{\text{tons}}{4\cdot5}}{11\cdot5}(7'\cdot85 + 2'\cdot91) = 4\cdot21 \text{ tons.}$$

The reaction at right bearing

$$= \frac{\overset{\text{tons}}{4\cdot5}}{11\cdot5}(8\cdot59 + 3\cdot65) = 4\cdot79 \text{ tons.}$$

The bending moment at A is therefore

$$\overset{\text{tons}}{4\cdot21} \times 3\cdot65 = 15\cdot37 \text{ foot-tons.}$$

The bending moment at B is therefore

$$\overset{\text{tons}}{4\cdot79} \times 2\cdot91 = 13\cdot94 \text{ foot-tons.}$$

The bending moment at A, at which point the maximum bending moment occurs, is therefore the only one which need be considered. In addition to the bending moment which we have found as above we have to add the bending moment at that point due to the weight of the trough itself, which we may take as $0\cdot75$ cwt. per foot run.

The bending moment at A due to the weight of the trough will therefore be $0\cdot21 \times 3\cdot65 - 0\cdot1$ $\times 1\cdot82$ tons $= 0\cdot53$ foot-tons.

The total bending moment at A is therefore $15\cdot37 + 0\cdot53 = 15\cdot90$ foot-tons $= 190\cdot80$ inch tons.

The moment of resistance of this section of trough has been worked out in the manner previously explained, and found to be $52\cdot0$, taking the inch as unit.

The maximum stress in floor will therefore be $\frac{190 \cdot 80}{52 \cdot 00} = 3 \cdot 67$ tons.

On looking at the way in which the trough is placed, it will be seen that this maximum stress is a tensile stress, and on looking at the diagram of "equivalent area," Plate II. Fig. 9, it will be seen that the maximum compressive stress is less than this, and is equal to $3 \cdot 67 \times \frac{3 \cdot 75}{4 \cdot 75} = 2 \cdot 9$ tons.

We see therefore that the stresses are within the limits which we have adopted.

It may be instructive here to consider the effect of placing this trough the other way up, viz. making the joint and cover in the bottom instead of the top of the trough, as is sometimes done. When the joint and cover are at top, we have the whole section of the trough and cover available; as, the rivets being in the compression portion, nothing need be deducted for rivet-holes. On the other hand, when the joint is at the bottom, two rivet-holes each have to be deducted from area of trough and cover. This deduction of course lessens the equivalent area section, so that the maximum stress is greater, and in this case the maximum stress is a compressive one, and furthermore, takes place on a portion of trough more liable to deformation than when the trough is placed as we have done.

We shall now return to the main girders.

The dead weight consists of the main girders

themselves, the permanent way and the weight of the troughs.

The permanent way, including rails, fastenings, chairs and sleepers, works out at about $1\frac{1}{2}$ cwt. per foot run for each track, and the troughing is about 7 cwt. per foot run for the total width of bridge. It will be noticed that owing to the rails not being central between the side and centre girders, the load on the central girder (Plate II. Fig. 2), from permanent way and rolling load is *more* than double that on each of the side girders. The total distributed load on central girder will be made up as follows:—

		Tons
Main girder	=	4·5
Permanent way, $2 \times \overset{\text{cwts.}}{1{\cdot}5} \times 32'\,6'' \times \frac{6{\cdot}12}{11{\cdot}5}$ ft. ..	=	2·6
Troughs, $32'\,6'' \times \frac{7}{2}$ cwt.	=	5·7
Rolling load, 2×40 cwt. $\times 32'\,6'' \times \frac{6{\cdot}12}{11{\cdot}5}$ ft. ..	=	69·2
Total	=	82·0

The bending moment at centre will be $\frac{82 \times 32{\cdot}5}{8}$ $= 333$ foot-tons.

The total stress in either flange at centre will be $\frac{333}{3} = 111$ tons.

The net area of bottom or tension flange at centre must therefore not be less than $\frac{111}{5} = 22{\cdot}2$ square inches.

We shall therefore make the flange up as below at centre.

	Net Area. Square inches.
2 angles, $4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{5}{8}''$	= 9·2
1 plate, $16'' \times \frac{1}{2}''$	= 7·0
1 plate, $16'' \times \frac{1}{2}''$	= 7·0
	23·2

In each angle one rivet-hole, and in each plate two rivet-holes are subtracted. Each rivet-hole for a $\frac{7}{8}$-inch rivet is taken as one inch diameter. The angles are made large, so as to give a good bearing for troughs, and to diminish any racking action in bottom boom owing to connection with troughs.

FIG. 21.

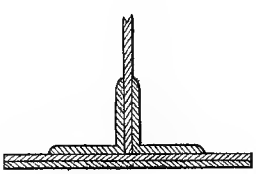
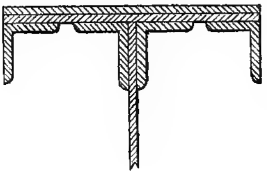


FIG. 22.



The gross area of the top or compression flange at centre must not be less than $\frac{111}{4\cdot25} = 26\cdot1$ square inches.

We shall make this flange up as below :—

	Gross Area. Square inches.
2 angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	= 6·5
„ „ „	= 6·5
1 plate, $16'' \times \frac{1}{2}''$	= 8·0
„ „	= 8·0
	29·0

In both flanges we have a little more section than is absolutely necessary.

In the bottom flange the two angles and one plate must of course extend for the whole length of the girder, but the second plate can be cut short. The simplest way of finding out the point at which the plate can be cut off is shown in Fig. 7, Plate II.

Let *a*, *b*, be the effective span, viz. 32 feet 6 inches to a scale of two feet to the inch, and *c*, *d*, be the total span. Erect a perpendicular at the centre, and lay off on it lengths on some scale equal to the areas of the different angles and plates of which the flange is made up. The scale we have taken is 40 square inches, equal to one inch. The different steps are clearly shown in the figure. Then lay off on the same perpendicular, and to the same scale, the total area required at the centre, viz. 26·1 square inches, which is represented by the length *o*, *e*. Through the points *a*, *e*, *b*, describe a parabola, and the curve will give the area required at each point of the flange, and therefore gives us the point at which we can cut off the outside plate. As the scales have been taken so that *o*, *e*, is less than one-tenth of *a*, *b*, we have described a circle instead of a parabola. In the figure it will be seen that the length of outside plate necessary scales 17 feet 2 inches, but it will be noticed that it is actually made longer. The reason of this is that the ends of the plate are prolonged to form a cover for the joints in the plate next the angles. This will be referred to

later on. A similar construction is shown for the top flange.

The maximum shear on the web will be at the abutment, and will therefore be half the total weight on the girder, viz. $\frac{82}{2} = 41$ tons.

The depth of web is 36 inches, from which we must subtract 6 inches for rivet-holes, leaving a net depth of 30 inches.

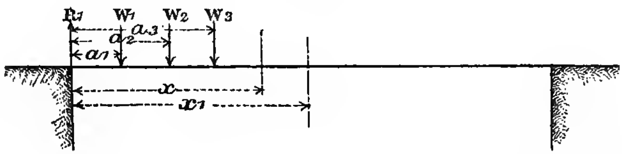
The allowable shearing stress per square inch is $3\frac{1}{3}$ tons, we therefore require 12·3 square inches of net area. We shall therefore make the web $\frac{1}{2}$ inch thick throughout, which gives a net shearing area of 15 square inches.

We have now settled the sections of flange plates and web plates. The usual pitch of rivets connecting the web to the flange is 4 inches, and we shall try if that is sufficient in the present case.

If we take the total stress in the flange at any two points, any distance apart, the amount of the increase of stress in the flange between the two points must be transmitted through the rivets connecting the angle bars of the flange to the web between these points. We can thus find out the shear on the rivets connecting the flange to the web. In the case of plate girders with horizontal flanges the shear is easily obtained from the fact that the horizontal shear per foot run at any point between the web and the flange is equal to the vertical shear per foot run in the web at the same point. This may be proved as follows :—

In Fig. 23 we have a girder supported at both ends. Take a section distant x from the left abutment, and let the reaction at the left abutment be R_1 and let the weights between the left abutment

FIG. 23.



and the section distant x from the abutment be W_1, W_2, W_3, &c., at the distances shown.

The bending moment at $x = R_1 x - W_1 (x - a_1) - W_2 (x - a_2) -$ &c. $= M_1$.

Take another section distant x_1 from the left abutment beyond x, and infinitely close to x, then,

The bending moment at $x_1 = R_1 x_1 - W_1 (x_1 - a_1) - W_2 (x_1 - a_2) -$ &c. $= M_2$.

Now if d be the depth of the girder, and if the direct flange strains due to M_1 and M_2 be denoted by T_1 and T_2, then,

$$T_1 = \frac{R_1 x - W_1 (x - a_1) - W_2 (x - a_2) - \text{\&c.}}{d},$$

and

$$T_2 = \frac{R_1 x_1 - W_1 (x_1 - a_1) - W_2 (x_1 - a_2) - \text{\&c.}}{d}.$$

The horizontal shear between x and $x_1 = T_2 - T_1$, and

$$T_2 - T_1 = \frac{R_1 (x_1 - x) - W_1 (x_1 - x) - W_2 (x_1 - x) - \text{\&c.}}{d}$$

$$= \frac{(x_1 - x)(R_1 - W_1 - W_2 - \text{\&c.})}{d}.$$

If we divide this by $x_1 - x$ we get the horizontal shear per foot run between x_1 and x, which is therefore,

$$\frac{R_1 - W_1 - W_2 - \&c.}{d}.$$

But when the points x and x_1 are infinitely close to each other this expression is also the vertical shear per foot run in the web at this point.

It is evident from looking at the curve of bending moments, that since the shear on the rivets connecting the flange to the web between any two points is equal to the difference of the stress in the flange between these points, that the maximum shear on these rivets occurs near the abutment, as the difference of stress at the extremities of a unit length of the flange is gradually increasing as we go from the centre to the abutment, and is a maximum at the abutment.

It will therefore be sufficient to try if we have enough shearing and bearing area of rivets connecting the web to the flange near the abutment.

In the 3 feet of the girder next to the abutment we have the average vertical shear = 37 tons. As the girder is 3 feet deep this is 12·3 tons per foot run, and this is also the shear per foot run between the flange and web.

With rivets of 4-inch pitch, we have three rivets in one foot, and have therefore 3 × 0·6 square inches = 1·8 square inches of area. All these rivets are in double shear; we have therefore 3·6 inches of

effective shearing area. The shearing stress per square inch is therefore $\frac{12 \cdot 3}{3 \cdot 6} = 3 \cdot 4$ tons, which is an allowable stress to adopt.

The bearing area of the rivets for a length of one foot is $3 \times \frac{7}{8}$ in. $\times \frac{1}{2}$ in. $= 1 \cdot 32$ square inches; the bearing stress per square inch is therefore $\frac{12 \cdot 3}{1 \cdot 32}$ $= 9 \cdot 3$ tons per square inch.

This stress is rather excessive, and it will therefore be better to increase the diameter of the rivets for 6 feet at each end of the girder to 1 inch, which reduces the bearing stress to 8 tons per square inch.

In the case of a comparatively shallow girder, heavily loaded, of which the girder we are considering is a good example, there is nearly always a difficulty about the rivets connecting the web to the flange near the abutments. With the ordinary $\frac{7}{8}$-inch diameter rivets and 4-inch pitch, the bearing area is nearly always deficient, and the shearing area very often is so. To overcome this, the pitch of the riveting can be diminished, or the size of the rivets increased for a short distance near the abutment. The former is the better method, but with a trough floor it is difficult to alter the pitch in the bottom flange, and in this case we have, therefore, increased the diameter of the rivets. Of course, other ways in which the bearing area can be increased are to increase the thickness of the web to $\frac{5}{8}$ inch near

the abutment, or to increase the depth of the girder itself.

We shall now go back to the outside girders, and the covers and details of these and the centre girder can be considered together afterwards.

The total distributed load on an outside girder (Plate II. Fig. 1) will be made up as follows :—

	Tons.
Main girder	3·5
Permanent way, $1' \, 5'' \times 32' \, 6'' \times \frac{5 \cdot 38}{11 \cdot 5}$ ..	1·2
Troughs, $32' \, 6'' \times \frac{7}{4}$ cwt.	2·9
Rolling load, 40 cwt. $\times \frac{5 \cdot 38}{11 \cdot 5}$	30·4
	38·0

The bending moment at centre will be

$$\frac{\overset{\text{tons}}{38 \cdot 0} \times 32 \cdot 5}{8} = 154 \text{ foot-tons.}$$

The total stress in either flange at centre will be $\frac{154}{3} = 51 \cdot 4$ tons.

The *net* area of the bottom or tension flange at centre must not be less than $\frac{51 \cdot 4}{5} = 10 \cdot 3$ square inches.

We shall therefore make this flange up at centre, as shown in Fig. 24, viz. :

	Square Inches.
2 angles, $4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{1}{2}''$	= 7·5
1 plate, $15'' \times \frac{1}{2}''$	= 6·5
Total	14·0

The *gross* area of the top or compression flange at centre must therefore not be less than $\frac{51 \cdot 4}{4 \cdot 25} = 12 \cdot 1$ square inches.

FIG. 24.

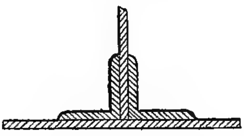
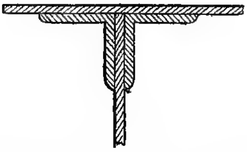


FIG. 25.



We shall therefore make this flange up at centre as shown in Fig. 25, viz.:

	Gross. Square Inches.
2 angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	= 6·5
1 plate, $15'' \times \frac{1}{2}''$	= 7·5
	14·0

In the case of the outside girder the section will be continuous all through.

It will be noticed that in this girder the section is rather heavier than necessary, particularly in the bottom flange. This is due to the heavy angles in the bottom flange, and to the desire of not putting less than $\frac{1}{2}$-inch thick plates in the flanges, when each flange has only one plate.

The shear at abutment is 19 tons, and we shall therefore require $\frac{19}{3\frac{1}{3}} = 5 \cdot 7$ square inches net section of web. With a $\frac{3}{8}$-inch web, our minimum thickness of web, we shall have 11 square inches.

The rivets connecting web to flange angles are evidently ample.

We shall now consider what covers are necessary

and what lengths of flange plates and web plates we shall adopt.

We shall deal with the central girder first.

We have already seen that as regards the top flange we might cut off the outside plate at 7 feet 3 inches on each side of the centre; but we have prolonged it on each side beyond this distance so as to act as covers for joints in the inner plate. The joint in the inner plate must come between two rows of rivets, and it will be seen that the riveting is arranged so that the rows of riveting are multiples of 4 inches from the centres of the stiffeners, and therefore in this particular case from the centre of the girder. The joint in the inner plate must therefore be at a distance from the centre which will be a multiple of 4 inches plus 2 inches. We shall therefore make it at a distance of 7 feet 6 inches on each side from the centre. Now we want to find out how far the outer plate must extend beyond the joint so as to have sufficient rivets to transmit the stress across the joint.

We have already shown (p. 57) that, for a compression flange, the total area of the rivets necessary to make up the section of the plate must be equal to the *gross* section of the plate. The gross section of a 16-inch by $\frac{1}{2}$-inch plate is 8 square inches, and the area of a $\frac{7}{8}$-inch diameter rivet is 0·6 square inch; therefore, the number of rivets required is $8 \div 0{\cdot}6 = 14$ rivets. The outer plate will therefore extend, as shown, 1 foot 4 inches

beyond the joint in the inner plate, and the whole length of the outer plate will therefore be 17 feet 8 inches.

For the bottom flange the joint is very similar. In a tension flange, so as to avoid loss of section owing to rivet-holes, four rivets are not put in a row as in a compression flange, but are arranged as shown. The joint is therefore generally made through the row of rivets in the angle bars, and will therefore be a multiple of 4 inches from the centre. We shall therefore make the joint in the inner plate at a distance of 8 feet 8 inches, on each side from the centre.

We have already shown (p. 56) that, for a cover for a tension flange, the total area of rivets necessary to take up the section of the plate must be equal to one and a half times the net area of the plate. The net area of the 16-inch by $\frac{1}{2}$-inch plate is 7 square inches, therefore the number of rivets required is $(7 \times 1\frac{1}{2}) \div 0{\cdot}6 = 18$ rivets. This number of rivets will necessitate a lap of 1 foot 10 inches of the outer plate, and the total length of the outer plate will therefore be 21 feet.

As regards the web we shall make a joint of 7 feet on each side of centre at the T stiffeners, and we must therefore find out the maximum shear on the web at this point. The simplest way to do this has already been explained (p. 15), and is now shown in Fig. 8, Plate II. A B is the effective span, viz. 32 feet 6 inches to a scale of half inch to the

foot. A D is the maximum shear at the abutment, due to the live load, on a scale of 20 tons to the inch, and is of course equal to half the live load, or 34·6 tons. A E is the maximum shear at the abutment due to the dead load, to the same scale, and is equal to 6·4 tons. Join E to the centre C, and through D draw a parabola, or in this case a circle, touching A B at B. For the half of the girder to the left of the centre the ordinates between C E and D B give the maximum shears at each point. The maximum shear, therefore, at a distance 7 feet from the centre, is F H, which scales 20 tons. The joint is covered by $\frac{3}{8}$-inch covers and T bars on each side, the section of which is of course much greater than that of the web. The allowable shearing stress on rivets is $3\frac{1}{3}$ tons per square inch, and the shearing area of rivets required is therefore $20 \div 3\frac{1}{3} = 6$ inches. With $\frac{3}{4}$-inch diameter rivets, this requires 15 rivets. The actual number of rivets in the joint as drawn is 12 (not counting the rivets in horizontal angle bars) and they are all in double shear, and therefore equivalent to 24 rivets in single shear, and the number of rivets in the joint is therefore sufficient as regards shearing area.

As regards bearing area, the bearing area of the 12 rivets is $12 \times \frac{3}{4} \times \frac{1}{2} = 4 \cdot 5$ square inches, and the bearing stress per square inch is $\frac{20}{4 \cdot 5} = 4 \cdot 5$ tons, which is very moderate. The joint is therefore amply strong in every particular.

For the outside girders it will not be necessary to go through the calculations for covers and lengths of plates, as the work would be similar to that already done for the central girder. It must be remembered that the positions of stiffeners and rivets in the outer girder really depend on the central girder, as they must fit in with positions of troughs which are fixed on the central girder.

On account of the skew of the bridge, the ends of an outer girder will not be similar as regards stiffeners and riveting. The two outer girders are, however, similar to each other, but their corresponding ends are reversed in position.

The top flange plate is made in two lengths. The lengths are made different so that the cover may not come where there is a slight difference in the ordinary run of riveting owing to the stiffeners. The bottom flange plate is jointed as in the central girder. The joint in the web is rather stronger than necessary, but it is more satisfactory to put in plate covers in addition to the T bars than to trust to the T bars alone.

It will be noticed that the ends of the central girder are rounded off. This is generally done, and the ends of the outside girders are sometimes rounded too.

All the lengths of angle bars in the girder can easily be obtained in one length, but in the case of those rounded off at the ends it will be better to cut them near the ends and put on covers, as the bending

of a bar of this length at both ends is an awkward job. The covers for these angles are not shown in the general elevation of the girder, but are shown in Fig. 10, Plate II. The angles connecting the web to the flange are fully covered by double covers, but the outer angles are not fully covered, as we have much greater section of flange near the abutment than is absolutely necessary.

Each girder has a $\frac{3}{4}$-inch bearing plate at the abutments, riveted to the underside of the bottom flange with countersunk rivets. The bearing plates for the centre and side girders differ slightly so as to fit in with the riveting. As a general rule it is not necessary to use cast-iron bed plates for spans under 40 or 50 feet, and in the present case the girder will rest directly on a smooth dressed hard stone. The maximum pressure on this stone at the ends of the central girder will not be more than about 16 tons per square foot, which is well within what may be put on most stones. If the bridge is on an incline the girders ought to be fixed to the masonry of the abutments at one end by means of holding-down bolts 7 or 8 feet long, built into the masonry with cast-iron anchor plates at their end, and coming up through holes in the bed stone.

As regards the positions and number of stiffeners necessary, of which as yet we have said nothing; a very simple plan is given by Rankine for determining these, based on the strength of the web to resist buckling treated as a column.

The author is of opinion, however, that stiffeners are generally put in without making any such calculations, their positions and number being based on previous experience. They are very seldom, except in girders of very short span, put more than 6 or 7 feet apart, and are of course much closer near the abutments than the centre. They have often to be fixed at certain distances on account of certain dispositions of troughs or cross girders. The web over the bearing at the abutment should be well stiffened with plate stiffeners, which also prevent any bending of the edge of the bearing plate and flange. Plate stiffeners in other parts of the girder, in addition to stiffening the web, are also useful as stiffening to a small extent the compression flange.

We have shown the troughs as resting on the angles of the bottom booms of the main girders to which they are riveted, and they are fixed on top by the stiffeners and by the longitudinal angles riveted to the web. These latter angles are often cut up into short lengths and the end of each trough fixed by small separate pieces of angles. This method, however, is not as good as using long lengths, as these short lengths tend to rack the web of the main girders locally. The continuous bars have, however, for purposes of erection, often to be cut into shorter lengths to facilitate getting the troughs into position. Two or three sections of troughs are often riveted together before being brought to site of erection to save outside riveting.

Packings between the troughs and the top angles give a little more room for moving the troughs into position, and are shown in the drawings of this bridge. The ends of troughs resting on the abutments can be built up solid under the flute with brickwork or concrete.

In the bridge we are considering the skew is slight, and we have therefore considered that there is no reduction in the load on the main girders due to this cause. With large skews, however, there will be a very considerable reduction in this respect, as a number of the troughs would be resting on the abutment at one of their ends.

The drainage of the floor is taken by gutters on each side of the central girder, which is bedded 1 inch lower than the side girders to give water a lead to the gutters.

Fig. 1, Plate III., shows a cross section of a similar bridge but with a ballasted floor. There is also a slight alteration in the flooring itself which is made of a patent cambered trough which has many advantages over the ordinary troughs. Owing to the increased depth in the centre, the trough is stronger just where the extra strength is required, and the rain water is all drained to the centre of the trough away from the main girders. There is the one disadvantage of having a hole in the tension flange at its centre, but this is made elongated so as to lose as little section as possible, and whatever is lost is more than compensated for by other advan-

tages. The gutter is fastened to the bottom of the trough by twisted hangers coming through the elongated hole, thus doing away with riveting it, or hangers to the bottom of the troughs.

This cambered flooring can of course also be used with advantage for non-ballasted bridges.

An open parapet is shown and is made 4 feet 6 inches above rail-level. This height of parapet is not absolutely required by law except in viaducts.

Fig. 2, Plate III., shows a cross section of a bridge with three main girders and a floor consisting of cross girders and light longitudinal troughs. The permanent way is laid in ballast with a minimum of 3 inches between the top of the troughs and the bottom of the sleepers.

The troughs are made in sections consisting of two flutes, and the sections are connected together by $\frac{3}{8}$-inch covers. The troughs are $4\frac{3}{4}$ inches deep, of $\frac{3}{8}$-inch metal, and 1 foot 4 inches pitch as shown. They can be obtained 30 feet long, and if longer lengths are required they can be jointed on a cross girder, and bent cover plates used. The cross girders are spaced 7 feet apart. The moment of resistance of one trough without cover plate is about 11·30 in inch units, and with cover about 12·90; the average of the two will be 12·10. If we assume that the load is distributed over seven troughs, the moment of resistance of the seven will be about 84·7, being a little more or less, as there are three or four troughs, with or without covers.

The maximum rolling load between two cross girders will be 17·5 tons, and if we assume that half of this is transmitted by one sleeper, and one-quarter by each adjacent sleeper, the bending moment on the trough at the centre, taking the sleepers at 2 feet 6 inches centres, will be,

Inch-tons.

$$\frac{17 \cdot 5}{2} \times 42 - \frac{17 \cdot 5}{4} \times 30 = 236$$

The dead load between two cross girders is about 2 tons, and the bending moment due to this will be $\frac{2 \times 64}{8} = 16$

Therefore the total bending moment is 252

Dividing this by the moment of resistance of the seven troughs, we get $\frac{252}{84 \cdot 7} = 3$ tons as the maximum stress per square inch on the troughs. This floor, including cross girders, is about the same weight as the floors already considered, but of course requires a little more headroom.

Figs. 3, 4 and 5 in Plate III. are cross sections of bridges which it has been found useful to adopt in certain cases.

Fig. 3 shows a cross section and part longitudinal elevation of a three-girder bridge where the headroom has been very limited. The sleepers are laid in the troughs and packed with ballast. This class of floor would of course only be used

under exceptional conditions, and is given more as a type to avoid than to adopt.

Fig. 4 shows a cross section of a bridge for spans of between 12 and 20 feet. Each rail is carried on a longitudinal sleeper laid in a trough, and a plate floor is riveted to the tops of the troughs and to a light parapet girder. This type is useful when headroom is limited, and this floor of four trough and longitudinal sleepers is often used for viaducts of a number of spans, the troughs acting as rail-bearers.

Fig. 5 shows a cross section of a bridge with two main girders and transverse troughs. A cross section of the troughs is also given. The rails are laid on longitudinal sleepers fastened by short lengths of angle iron to the top of the troughs.

Fig. 6 is a cross section of a bridge of about 20 feet span with four main girders and a light parapet girder. Cross sleepers laid on a minimum of 3 inches of ballast are shown, but without ballast longitudinal sleepers fastened to the top of the main girders are often adopted. The ends of the main girders fit into recesses left in the masonry of the abutments. When this type of bridge is adopted for larger spans, the main girders are braced together by frames of diagonal and horizontal bracing at intervals.

Fig. 7 shows cross sections of various troughs which have been adopted and which can be obtained in various depths and of different widths of flutes.

The great majority of trough sections are the patents of different makers. Several other kinds in addition to those shown have been brought out, but we have shown those in most general use. In transverse trough flooring with three main girders that shown in bridge in Plate III. Fig. 1, is probably one of the cheapest and most efficient floors, and for longitudinal trough flooring with cross girders that shown in Fig. 2, Plate III. All the troughs shown are used extensively for spans under 20 feet without any main girders, the troughs being laid longitudinally from abutment to abutment and a light parapet girder adopted.

The principal points to keep in view in selecting trough flooring are (1st) strength in proportion to weight; (2nd) a minimum of riveting; (3rd) water-tightness; and (4th) facility in erection.

CHAPTER XII.

TWO MAIN GIRDER BRIDGE.

WE shall now consider the bridge shown in Plate IV. The clear span is 60 feet on the skew, and the bridge consists of two main girders, cross girders, rail-bearers, and ballasted buckle plate floor. The bridge is on a considerable skew, one of the main girders sitting 23 feet in front of the other. The spacing of the cross girders is the first matter for consideration.

In considering the rolling load, which is the principal load on the cross girders, the first thing to be noticed is, that however close the cross girders are spaced, we must at one time or another while a train is crossing the bridge, have the weight on the heaviest axle borne directly by each cross girder. It is evident, therefore, that without increasing the stress, which will come on the cross girder, due to the rolling load, we can space them so far apart that the rolling load brought on to them by the rail-bearers shall not exceed the direct load on them due to the heaviest axle loads. This will generally happen if the cross girders are spaced something less than the wheel base of two pairs of coupled

driving wheels apart. This distance would therefore be the *minimum* distance apart which the cross girders ought to be spaced in ordinary cases. There are, however, other points to be considered which make it desirable and economical to space the cross girders further apart than this minimum. The fewer cross girders we have the more economically the weight can be brought to the main girders, but again we get a limit to their distance apart owing to the increasing span, and consequently increasing weight per foot run of the rail-bearers. Another point to be considered is how the cross girders will come with regard to the centre of the main girders, as it is obvious that it will be well not to have a cross girder in the centre of a main girder. In skew spans the amount of skew is a principal factor in the spacing of the cross girders, as it is desirable to keep both main girders exactly the same (except that of course they are reversed in position) and at the same time not put the cross girders in the centre of the main girder.

"In ordinary cases it may be considered as generally advantageous to space the cross girders at distances equal to about one and one-half times the wheel base of two pairs of coupled driving wheels, or say from 9 to 12 feet, and to place rail-girders between the cross girders under each rail." *

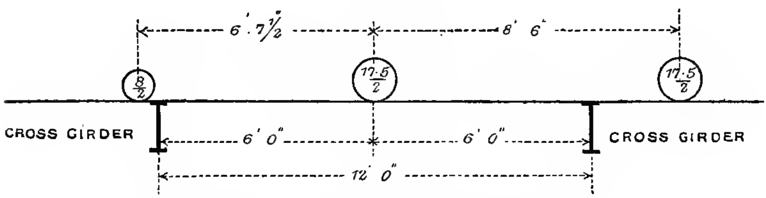
In the present case, 12 feet appears to be an economical distance to space the cross girders apart,

* B. Baker, 'Short Span Bridges.'

as at this distance our rail-bearers still remain comparatively light, and the centre of each main girder is practically central between two cross girders.

We shall first consider the rail-bearers. We shall make these 1 foot 6 inches deep as shown in Plate IV. The position of the rolling load which will give the maximum bending moment will be when the heaviest loaded axle is in the centre of the rail-bearer as below :—

FIG. 26.



The bending moment at the centre		Foot-tons.
due to this weight is $\frac{17 \cdot 5}{4} \times 6 \cdot 0$		$= 26 \cdot 25$

The dead load on a rail-bearer is as below :

	Tons.
Rail-bearer	$= 0 \cdot 4$
Floor	$= 0 \cdot 5$
Permanent way	$= 0 \cdot 4$
Ballast	$= 2 \cdot 0$
	$3 \cdot 3$

The bending moment at the centre due to this weight is $\frac{3 \cdot 3 \times 12}{8}$ $= 5 \cdot 00$

Total bending moment at centre $= 31 \cdot 25$

For purposes of calculation it will be close enough to take the effective depth as 1·25 feet.

Therefore the stress in either flange $= \dfrac{31 \cdot 25}{1 \cdot 25}$

$= 25$ tons.

For the bottom or tension flange we must therefore have a net area of not less than $\dfrac{25}{4 \cdot 5}$

$= 5 \cdot 6$ square inches.

We shall make this up as shown in Fig. 27, viz.:

Net Area.

2 angles, $4'' \times 3\frac{1}{2}'' \times \frac{1}{2}'' = 6 \cdot 00$ square inches.

For the top or compression flange we must not have less gross area than $\dfrac{25}{3 \cdot 85} = 6 \cdot 5$ square inches.

We shall make this up as shown in Fig. 28, viz.:

Gross Area.

2 angles, $4'' \times 3\frac{1}{2}'' \times \frac{1}{2}'' = 6 \cdot 5$ square inches.

FIG. 27.

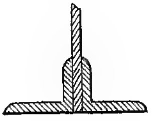
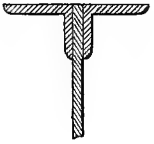


FIG. 28.



The maximum shear on the rail-bearers from the rolling load does not occur at the same time as we have the maximum bending, but will be when the loads are as shown in Fig. 29.

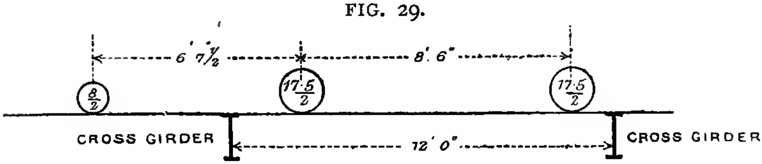
The shear at either end due to the

		Tons.
rolling load will therefore be $\frac{17 \cdot 5}{2}$	=	$8 \cdot 75$
and that due to the dead load will be $\frac{3 \cdot 3}{2}$	=	$1 \cdot 65$
Total shear		$10 \cdot 40$

Therefore the shear per foot run will be $\frac{10 \cdot 4}{1 \cdot 50}$ = 7 tons.

A $\frac{3}{8}$-inch web is more than ample for this.

FIG. 29.



As regards the rivets connecting the web to the flange angles, the shear per foot run is of course the same, viz. 7 tons per running foot, and if we use $\frac{7}{8}$-inch diameter rivets, space 3 inches apart, we have as shearing area, the rivets being in double shear, $2 \times 4 \times 0 \cdot 6 = 4 \cdot 8$ square inches, and therefore the shearing stress per square inch is $\frac{7}{4 \cdot 8} = 1 \cdot 5$ tons.

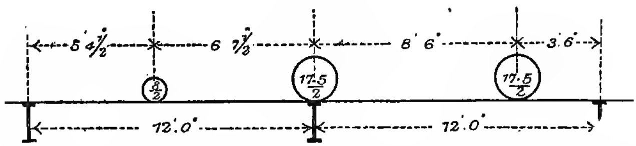
The bearing area will be $4 \times \frac{7}{8} \times \frac{3}{8} = 1 \cdot 3$ square inches, and therefore the bearing stress per square inch $= \frac{7}{1 \cdot 3} = 5 \cdot 4$ tons.

It is evident the rivets connecting the rail-bearers to the cross girders are ample.

We now come to the ordinary cross girders which we have made 2 feet 6 inches deep.

The greatest rolling load which can be brought on to the cross girder will be when the loads are as below:

FIG. 30.



The sum of the direct rolling load and the rolling loads brought on the central cross girder in Fig. 30 is 13·1 tons.

The dead load on the cross girder at each point of attachment of a rail-bearer is,

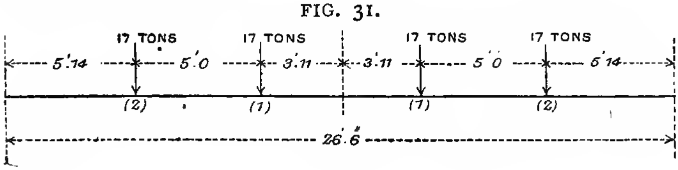
		Tons.
Load from cross girder itself	=	0·6
Load brought on by rail-bearer	=	3·3
		3·9

Therefore the total load at each point of attachment of rail-bearer is $3·9 + 13·1 = 17$ tons.

We have assumed in the above, for the sake of convenience, that the weight of the cross girder itself is concentrated at each of the points of attachment of the rail-bearers.

The total weight on each cross girder will therefore be as below:

FIG. 31.



The bending moment at the centre or points marked (1)

$$= 34 \times 10 \cdot 14 - 17 \times 5 \cdot 0 = 259 \cdot 75 \text{ foot-tons.}$$

That at points marked (2) is

$$34 \times 5 \cdot 14 = 174 \cdot 75 \text{ foot-tons.}$$

The stress in either flange at centre will be $\frac{259 \cdot 75}{2 \cdot 5} = 104$ tons.

The net area of tension flange at centre must therefore not be less than $\frac{104}{4 \cdot 5} = 23$ square inches.

We shall therefore make this up as shown in Fig. 32, viz.:

	Net. Square Inches.
2 plates, $16'' \times \frac{9}{16}''$	= 15·70
2 bars, $4'' \times 4'' \times \frac{5}{8}''$	= 8·00
	23·70

The gross area of the compression flange at centre must not be less than $\frac{104}{3 \cdot 8} = 27 \cdot 4$ square inches.

FIG. 32.

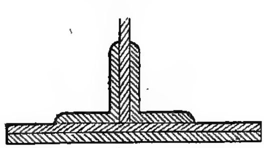
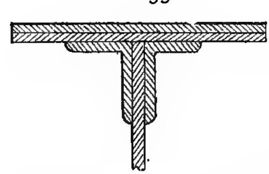


FIG. 33.



We shall therefore make it up as shown in Fig. 33, viz.:

	Square Inches.
2 plates, $16'' \times \frac{9}{16}''$	= 18·0
2 bars, $4'' \times 4'' \times \frac{5}{8}''$	= 9·2
	27·2

Fig. 1, Plate IV., shows curves of bending moments for this cross girder, and shows where the flange plates are cut off in a similar way to that already described in a previous bridge.

The maximum shear towards the end of the cross girder is 34 tons, and if we use a $\frac{1}{2}$-inch web throughout the shear per square inch on the web will be $\frac{34}{15} = 2 \cdot 3$ tons per square inch.

The horizontal shear per foot run between the flange and the web will be 13·6 tons. If we use $\frac{7}{8}$-inch diameter rivets with 3-inch pitch, the shearing stress per square inch on the rivets, remembering that the rivets are in double shear, will be

$$\frac{13 \cdot 6}{4 \times 0 \cdot 6 \times 2} = 2 \cdot 9 \text{ tons.}$$

The bearing stress per square inch will be

$$\frac{13 \cdot 6}{4 \times \frac{7}{8} \times \frac{1}{2}} = 7 \cdot 8 \text{ tons per square inch.}$$

This stress is more than allowable, so we shall increase the diameter of rivets between the outside rail-bearers and the main girders to 1 inch, and the bearing stress per square inch will then be

$$\frac{13 \cdot 6}{4 \times 1 \times \frac{1}{2}} = 6 \cdot 8 \text{ tons per square inch.}$$

All plates and angles in cross girders or rail-bearers can be obtained of the lengths required in steel, so that no covers will be necessary. If in iron, a joint would be necessary in the centre of web of cross girder covered with two $\frac{1}{4}$-inch plates. The

attachments of the longitudinals to the cross girders, and of the cross girders to the main girders, are shown clearly in the plate.

It will not be necessary to go through the calculations for the two short end cross girders, as there is no difficulty in finding out the weights on them Their construction is shown in the plate.

We now come to consider the rolling load to be adopted for the main girders when the load is brought on to them by cross girders spaced some distance apart, as in the bridge now under consideration.

The rolling load per foot run which we have got in the table on p. 31 for different spans, is obtained by reducing loads concentrated at certain points to equivalent uniform loads per foot run, and the reasons given there will also apply to reducing the loads concentrated at the junction of the cross girders with the main girders to a uniform load per foot run. May we therefore use the uniform loads per foot run given on p. 31 for the rolling loads on main girders of bridges with cross girders? This obviously depends on the spacing and position of the cross girders. If the cross girders are spaced a very long distance apart and one comes in the centre of the main girder, it is evident that the load per foot run as given in the table ought to be increased, and if the centre of the main girder is between two cross girders it ought to be diminished. In bridges of such spans as we are considering, and with cross

girders spaced from 8 to 12 feet apart, the variation from the equivalent uniform load given in the table may be about 8 per cent. on each side in extreme cases. A little practice will enable the amount to be estimated very closely. If extreme accuracy is required, it is easy to find the position of one or more locomotives which will give the maximum stresses on the main girders, but as a general rule there will be no necessity for such refinement unless there is some reason for saving every pound of metal possible. Of course we must remember that much higher rolling loads come on any *cross girder* than that due to an equivalent distributed load, but that, although in any moment during the passage of a train some cross girders transmit these heavier loads to the main girders, other cross girders transmit lighter loads, so that the whole effect on the main girder is nearly that due to the equivalent uniform distributed load.

If we now return to the main girder under consideration, we find from the table that the equivalent uniform distributed load for each track for a span of 65 feet is about 32·3 cwt. per foot run. We shall assume that, on account of the favourable position of our cross girders, this would be reduced by about 6 per cent., bringing it down to 30·5 cwt. per foot run for each track. Each cross girder will therefore bring its proportion of the rolling load at this rate on to the main girders.

In addition to the rolling load each cross

girder brings on to the main girders the weight of the cross girder itself, and its proportion of the weight of rail-bearers, floor, permanent way and ballast.

For convenience, we shall also assume that the weight of the main girder itself is concentrated at the points where the cross girders are attached to the main girders.

The weight at each of these points, with the exception of the two end cross girders, will be as below:

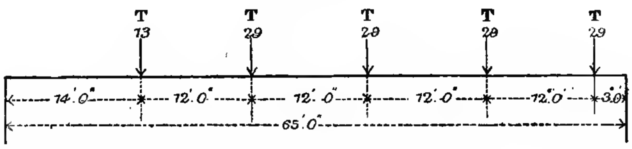
		Tons.
Rolling load, $\frac{30{\cdot}5 \times 12{\cdot}0}{20}$	.. .. ..	= 18·3
Main girder, $\frac{9 \times 12}{2 \times 20}$	.. ..	= 2·7
Cross girders, $\frac{2{\cdot}5}{2}$	.. ..	= 1·3
Rail-bearers	.. ..	= 0·8
Floor	.. ..	= 1·0
Permanent way	.. ..	= 0·8
Ballast	.. ..	= 4·0
		28·9, say 29

The weight brought on to each main girder by each end short cross girder will be as below:

		Tons.
Rolling load, $\frac{30{\cdot}5 \times 9}{20 \times 2}$	.. ..	= 6·9
Main girder, $\frac{9 \times 13}{2 \times 20}$	.. ..	= 2·9
Cross girder	.. ..	= 0·4
Rail-bearers	.. ..	= 0·5
Floor	.. ..	= 0·6
Permanent way	.. ..	= 0·3
Ballast	.. ..	= 1·4
		13·0

The loads and their positions on the main girder will now be as below. The effective span from centre to centre of bearings is 65 feet.

FIG. 34.



The reaction at the left abutment is .. 47·7 tons, and that at the right abutment .. .. 81·3 tons.

The bending moments at the points of loading, starting from the left abutment, work out at

667·8; 1084·2; 1152·6; 871·5; 243·4 foot-tons. At the point of maximum stress, therefore, viz. at a point 38 feet distant from the left abutment, the bending moment is 1152·6 tons.

If we make the depth of the girder 7 feet 6 inches, the stress in either flange at this point will be $\dfrac{1152 \cdot 6}{7 \cdot 5} = 153 \cdot 7$ tons.

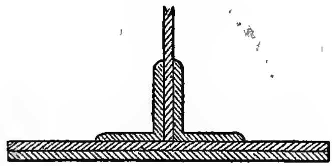
The net area of the tension flange must therefore be not less than $\dfrac{153 \cdot 7}{5 \cdot 35} = 28 \cdot 7$ square inches.

We shall therefore make this flange up as shown in Fig. 35, viz.:

	Net. Square Inches.
2 angle bars, 4″ × 4″ × ½″	= 6·50
1 plate, 1′ 8″ × ⅝″	= 11·25
1 plate, 1′ 8″ × ⅝″	= 11·25
	29·00

The gross area of the compression flange must not be less than $\frac{153 \cdot 7}{4 \cdot 65} = 33 \cdot 00$ square inches.

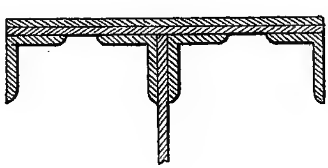
FIG. 35.



We shall therefore make this flange up as shown in Fig. 36, viz.:

	Gross. Square Inches.
2 angle bars, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	= 6·5
2 angle bars, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	= 6·5
2 plates, $1' \, 8'' \times \frac{1}{2}''$	= 20·00
	33·00

FIG. 36.



The points where the outside plates can be cut off are easily found by plotting the curve of bending moments to any scale, and dividing up the maximum ordinate proportional to the amounts of areas of plates and angles required, as shown in Plate IV., Fig. 2.

The maximum shear at the abutment to which the heavier load is brought, is 81·3 tons; with a half-

inch web, this gives a shearing stress on the web of $\frac{81 \cdot 3}{34 \cdot 0} = 2 \cdot 4$ tons per square inch of net section.

The stress per foot run between web and flange is $\frac{81 \cdot 3}{7 \cdot 5} = 10 \cdot 8$ tons.

With $\frac{7}{8}$-inch rivets at 4 inch pitch the shear on the rivets connecting the flange to the web will be, remembering that they are in double shear, $\frac{10 \cdot 8}{2 \times 3 \times 0 \cdot 6} = 3$ tons per square inch, and the bearing stress on the same rivets will be $\frac{10 \cdot 8}{3 \times \frac{7}{8} \times \frac{1}{2}} = 8 \cdot 3$ tons per square inch, the allowable stress being $8 \cdot 25$ tons.

We cannot get plates for the web 7 feet 6 inches deep without paying heavy extras, so we shall take plates 4 feet wide, and have 3 joints between each cross girder.

We shall require a $\frac{1}{2}$-inch web at only one of the abutments. In other portions of the girder it can be reduced to $\frac{7}{16}$ inch and $\frac{3}{8}$ inch as shown in Plate IV. Near the same abutment it will be necessary to have double lines of riveting in the web covers, at other places covers with single rows of riveting will be sufficient. The covers are heavier than actually necessary as they also act as packings in some cases. These details can be worked out in the way adopted for the central girder in Plate II., except that we must remember that in this case the shear is practically constant from cross girder to cross girder.

The joints in the flange plates are fully shown. The net area of the 1 foot 8 inch by $\frac{5}{8}$ inch flange plate in the bottom boom is 11·25 square inches, the number of $\frac{7}{8}$ inch diameter rivets required is therefore $11 \cdot 25 \times 1\frac{1}{2} \div 0 \cdot 6 = 28$ rivets. One joint in the inner plate is covered by producing the outer plate and making it act as cover and 30 rivets are given. The other joint in the inner plate is taken together with the joint in the outer plate and both are covered by one plate 1 foot 8 inches wide by $\frac{1}{2}$ inch thick, and two inside strips each $5\frac{1}{2}$ inches wide and $\frac{1}{2}$ inch thick, their united net area being 13·50 square inches, in comparison with 11·25 square inches, the net area of either $\frac{5}{8}$ inch plate. Some of the rivets by this grouping of the two joints are placed in double shear and a saving in material thus effected. The gross area of the 1 foot 8 inch by $\frac{1}{2}$ inch flange plate in top boom is 10·0 square inches. The number of $\frac{7}{8}$ inch diameter rivets required will there-be $10 \cdot 0 \div 0 \cdot 6 = 17$ rivets.

One of the joints of the inner plate is covered by the extension of the outer plate, and the other joint in inner plate, together with the joint in outer plate, is taken by one cover 1 foot 8 inches wide by $\frac{1}{2}$ inch thick.

Joints in angle bar can be made where necessary.

The joints of the floor plates are covered by T bars so as to have the plating continuous, and thus able to take up wind pressure.

The total wind pressure on the bridge with a

train running over it and a gale with a pressure of 56 lbs. to the square foot blowing, would be

$$\frac{65' \times 17' \times 56}{2240} = 28 \text{ tons.}$$

This must be considered as a uniformly distributed load and its moment is therefore $\frac{28 \times 65}{8} =$ 228 foot-tons. The depth of the horizontal girder, viz. the width of the bridge, is 26 feet 8 inches and therefore the stress in either flange of the horizontal girder due to the wind pressure would be

$$\frac{228}{26 \cdot 67} = 8 \cdot 5 \text{ tons.}$$

If we assume that only the bottom flange of one of the main girders would take this up it would be an increase of $\frac{26 \cdot 75}{8 \cdot 5} = \frac{1}{3}$ of a ton on the leeward bottom flange, which would not be very serious, as occasions on which a 56 lb. gale would be blowing, and two trains running over the bridge, would occur very seldom, if at all.

We have shown the main girders with $\frac{3}{4}$ inch bearing plates with countersunk rivets resting on planed cast-iron bed plates 2 inches thick. The maximum pressure on the masonry will be about 12 tons per square foot.

The free ends of the short cross girders and rail-bearers have bearing plates with countersunk rivets, resting on bed stones.

We ought to give the main girders a camber of about $1\frac{1}{4}$ inches.

It must be remembered in skew spans that a cross girder will not come at points of corresponding camber in each main girder. It will, therefore, often be better to keep all the cross girders horizontal by inserting packings underneath the bearing of the cross girders on the main girders; an allowance will also have to be made for this between the top of the cross girder and the stiffeners. These packings are not shown in the plate.

CHAPTER XIII.

CONCLUSION.

In concluding these remarks on Plate-Girder Railway Bridges, the author would wish to impress the necessity of not looking too closely on the theoretical side of the subject. The saving which may be obtained by trying to exactly proportion sections to stresses may often be more than lost by defects of a practical nature. The necessity of not using a great variety of sections in a bridge has been previously referred to, and it can be easily understood that a bridge builder has to pay higher prices to steel manufacturers for a quantity of different sections than when as many sections as possible are kept the same. The necessity of only using the ordinary market sections of steel and iron is a point which cannot be too strongly remembered, and this particularly is so in the case of short span bridges. In bridges of long span it will of course be possible to get material of special section at little more than ordinary rates if a large quantity is required. Smith's work, such as bending, joggling, &c., is very expensive, and should be reduced to a minimum.

For the above reasons it may often be found necessary and better to have sections a little heavier than what may be required by theory. Attention may also be drawn to the necessity of avoiding wide unstiffened flanges, particularly compression flanges. Local conditions ought always to be considered, particularly with regard to the method to be employed for erecting a bridge, and if the erection is kept in view while designing joints and connections a good deal of time and money may be saved later on. In connection with local conditions it would hardly be fair to consider a bridge close to a large town where a two or three minute service of trains passes over it as undergoing the same amount of wear and tear as a bridge in the country with, say, a half-hourly service. In the former case it would be well and desirable to have the bridge slightly stronger than in the latter, and particular attention should be paid to all floor joints and connections, in addition to such matters of detail as bearing stress on rivets, so that all racking action may be reduced to a minimum.

All parts should be designed as far as possible so as to be capable of being inspected and painted at intervals, thus helping to prolong indefinitely the life of the structure.

The Plates which have been given refer of course to railways in Great Britain, and it will be understood that in new countries the expense of iron or steel floors is not generally incurred. In the greater

portions of Canada, the United States, and many colonies, the greater portion of the floors of bridges up to the present time have been of timber, but in a number of new bridges plate floors are now being adopted.

The weight of wrought iron is 40 lbs. for a plate one inch. thick and one foot square, and steel is two per cent. heavier. An allowance of three to four per cent. is generally added to the weight of plates and bars in any bridge to cover the weight of rivet heads.

It is rather difficult to say much as regards the cost of bridges, as it is dependent so much on the distance which the bridge is from the manufacturer's yard, and on the difficulties of erection. At the present time, however, it may be considered that steel bridges for new railways can be erected complete at from 13*l.* 0*s.* to 16*l.* 10*s.* per ton, dependent on the conditions before referred to. In the case of replacing old bridges on a line where the traffic has to be kept up all the time, these prices might of course be very much exceeded.

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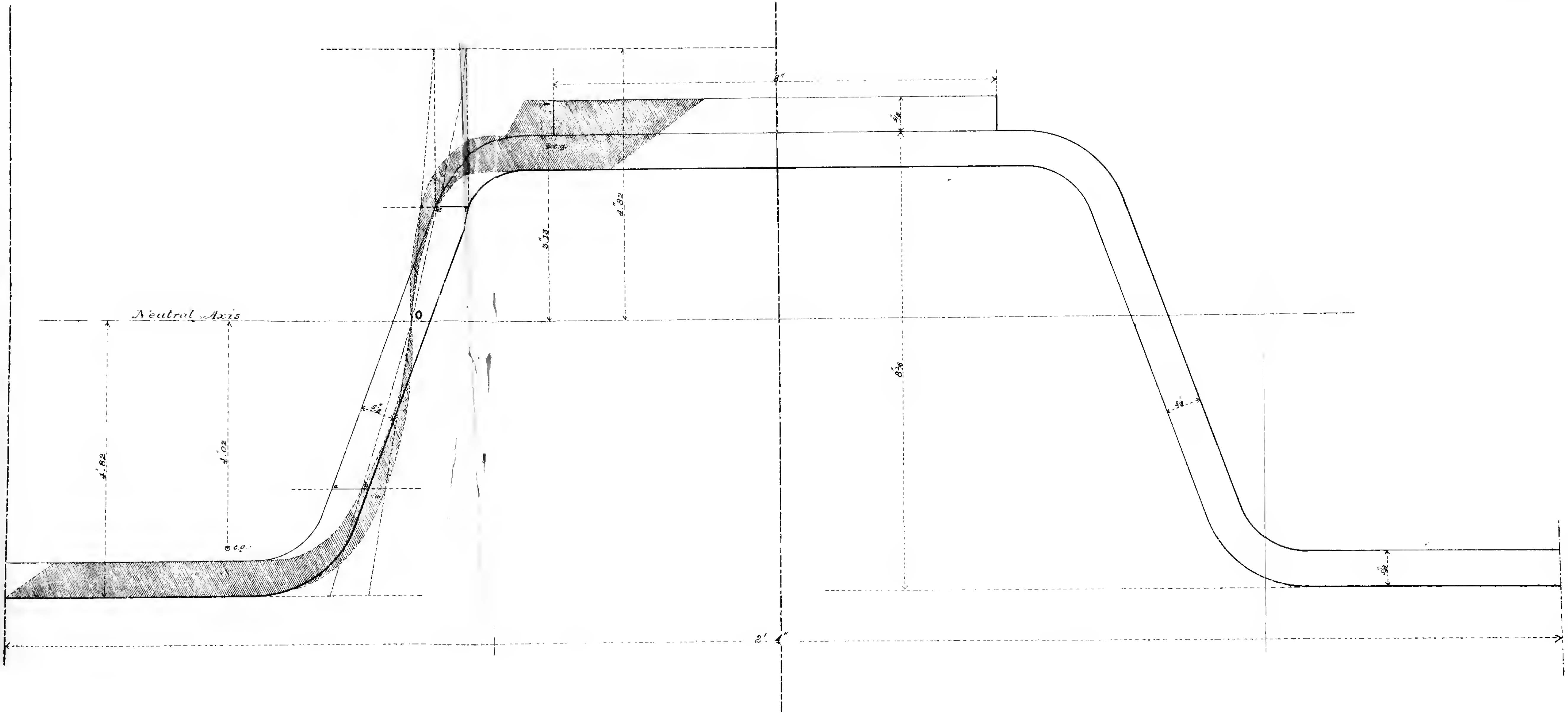

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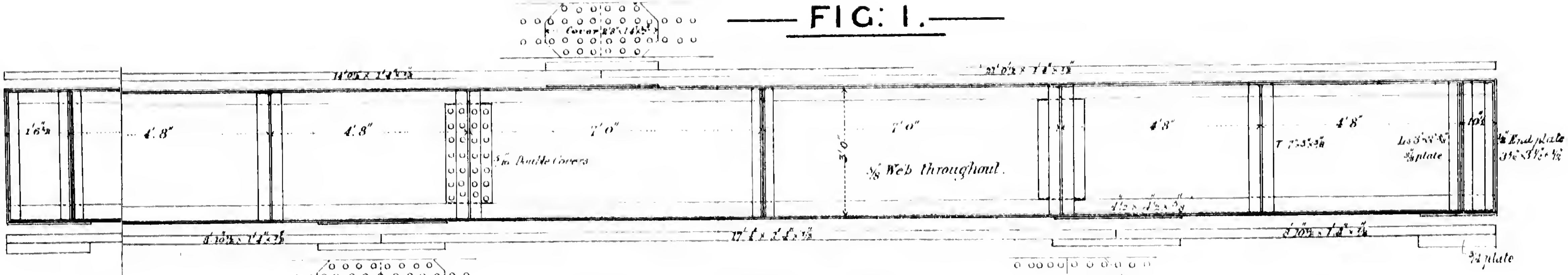

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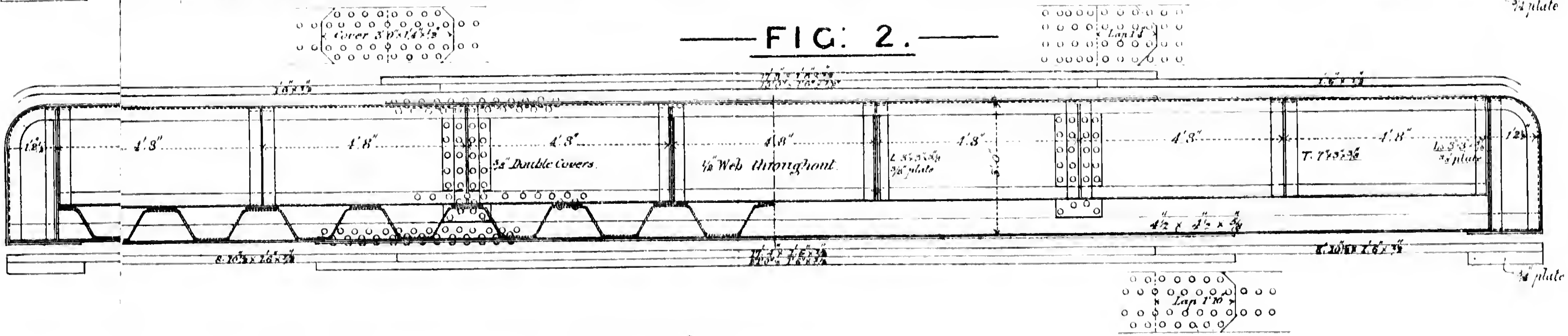

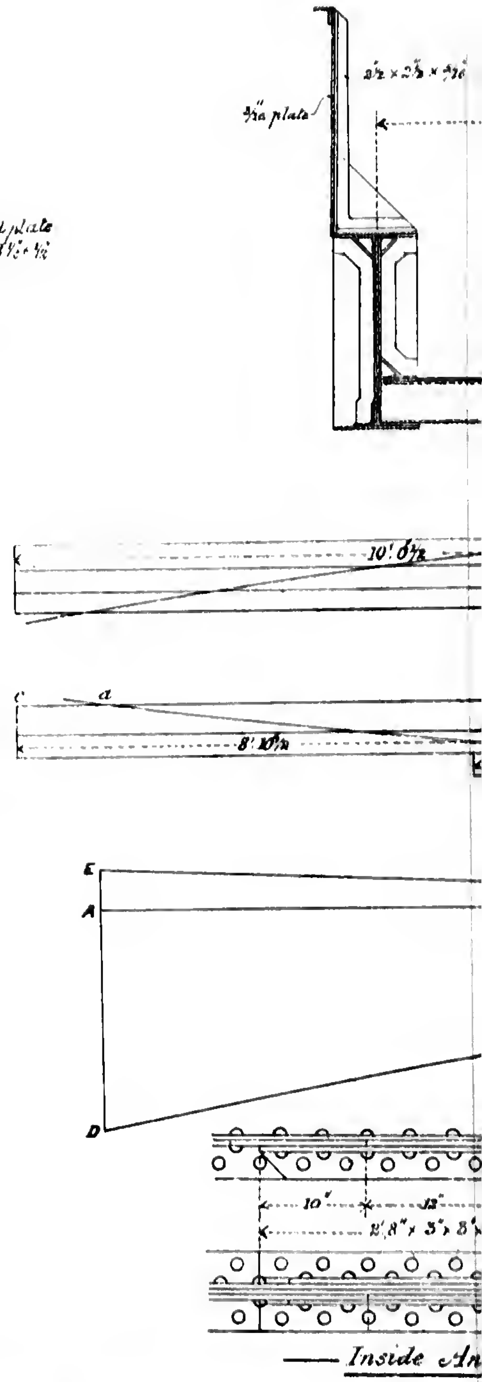

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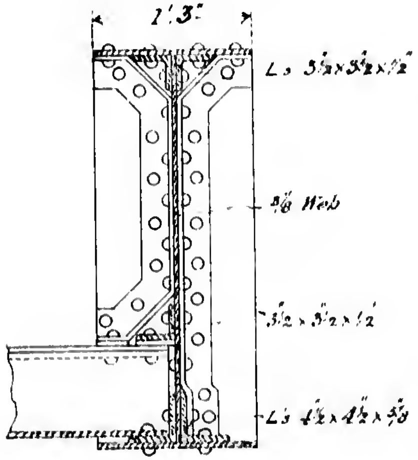

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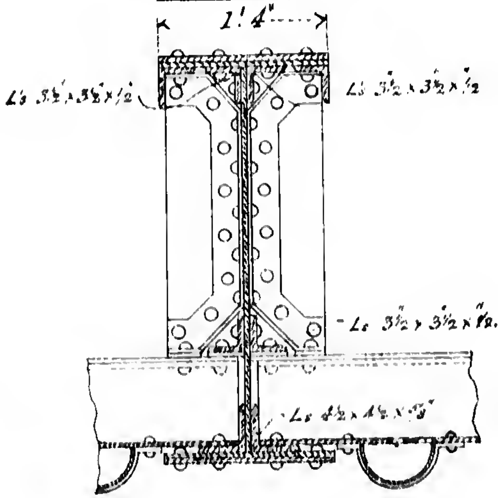

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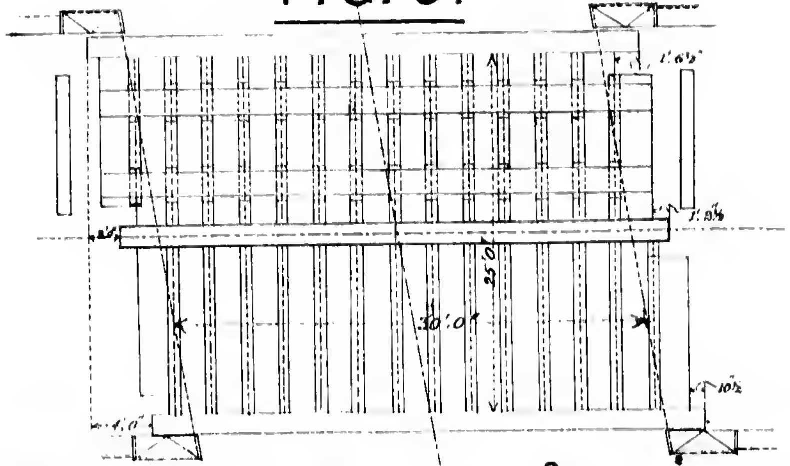

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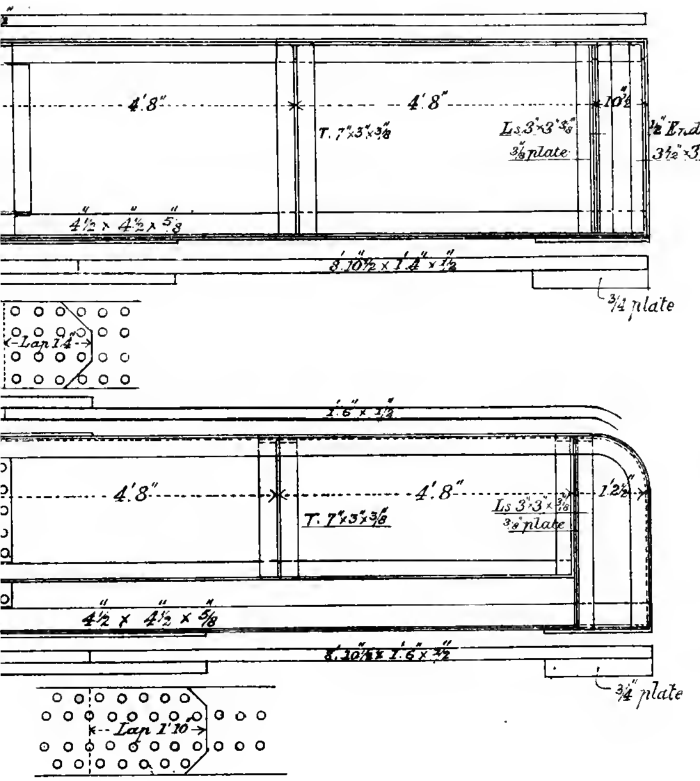

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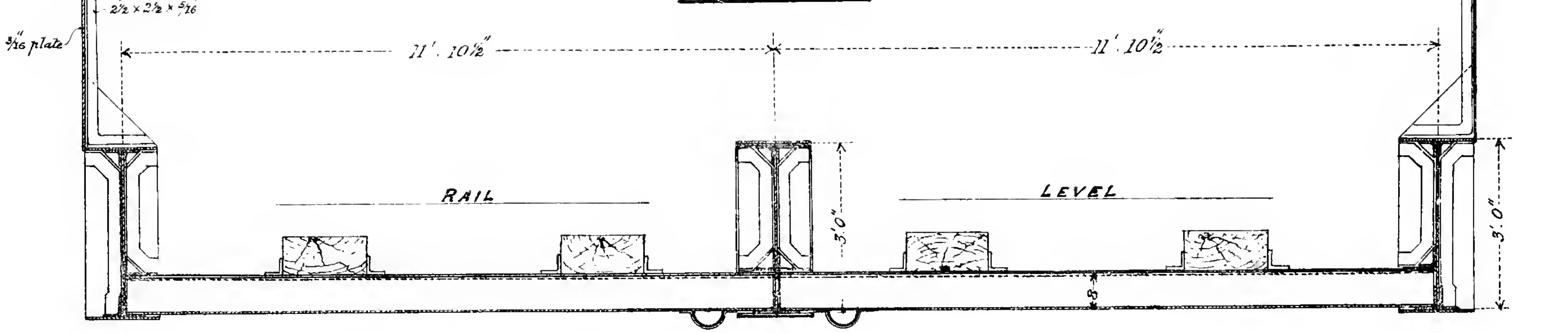

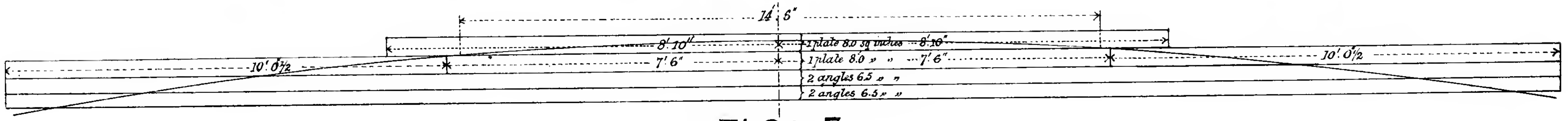

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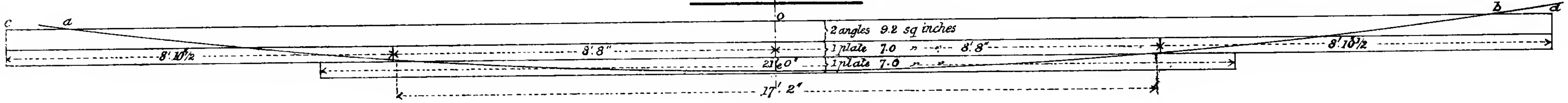

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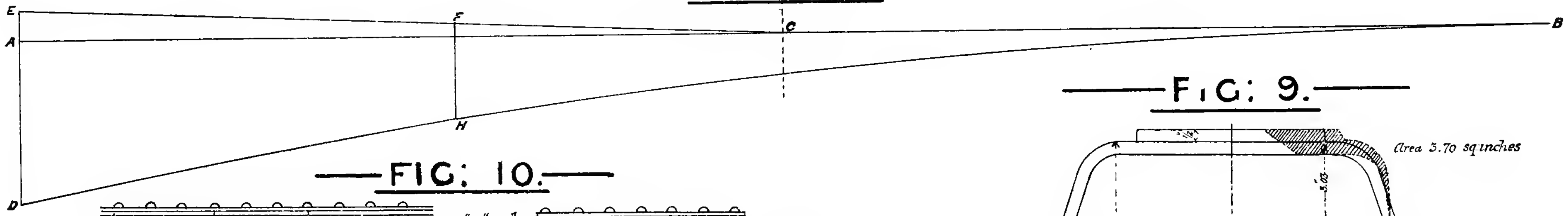

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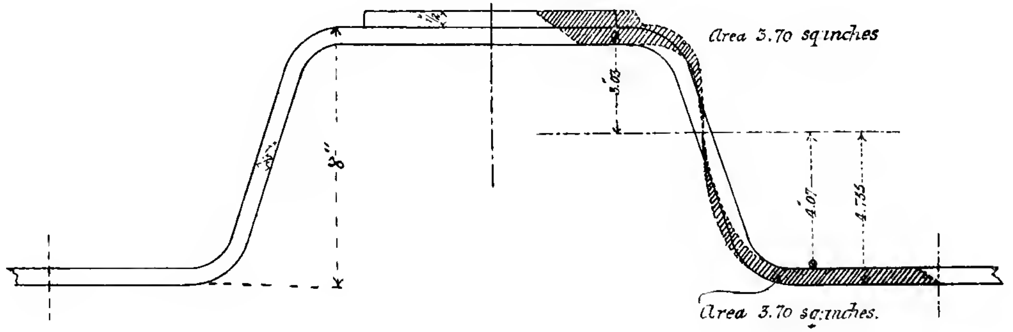

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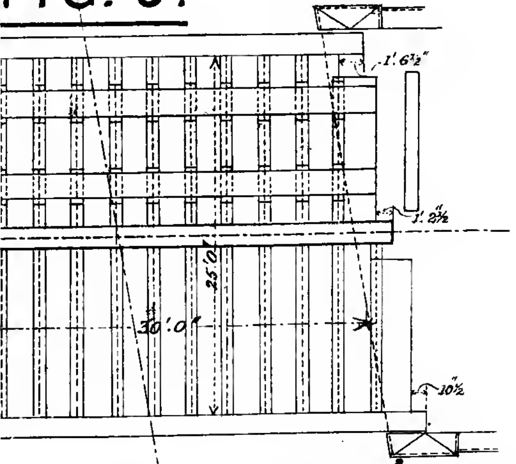

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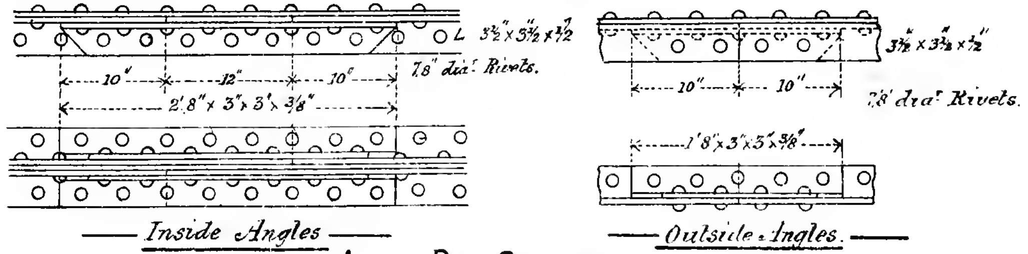
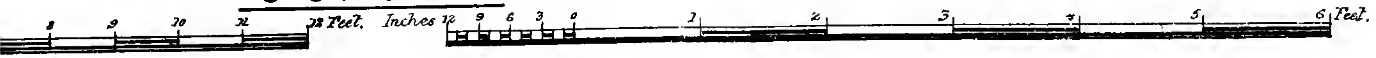


ANGLE BAR COVERS

SCALES.



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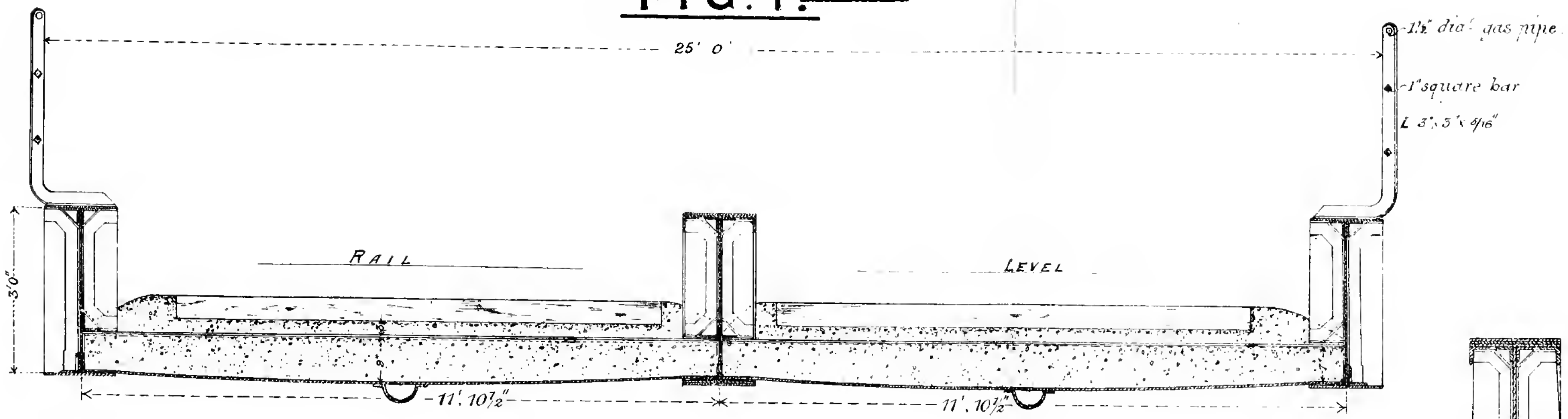

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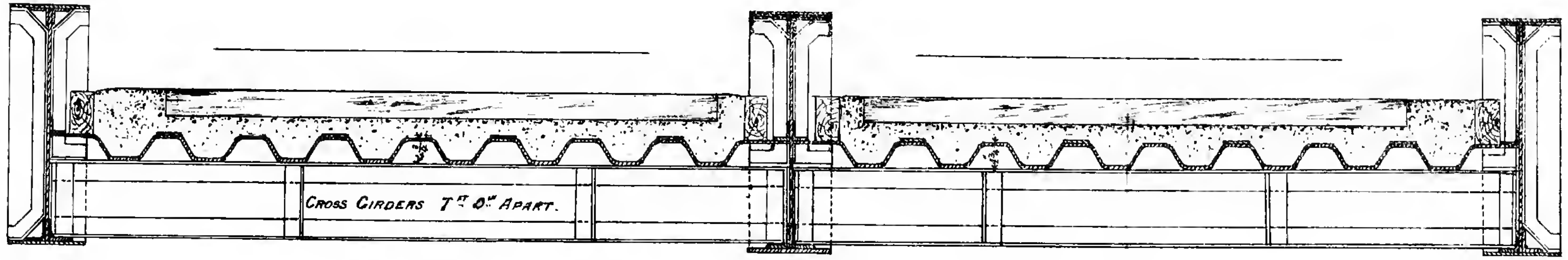

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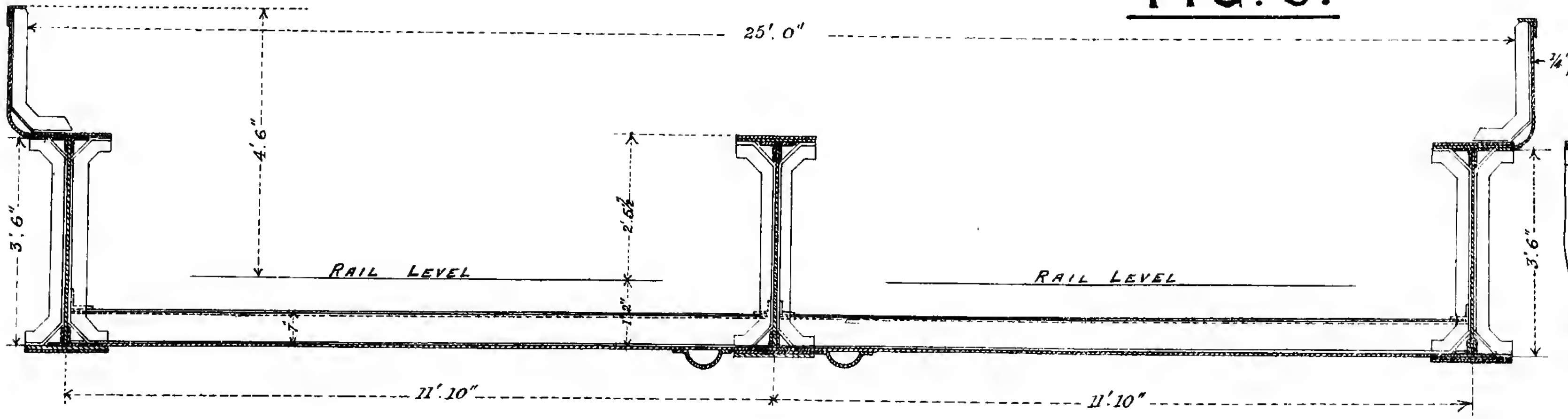

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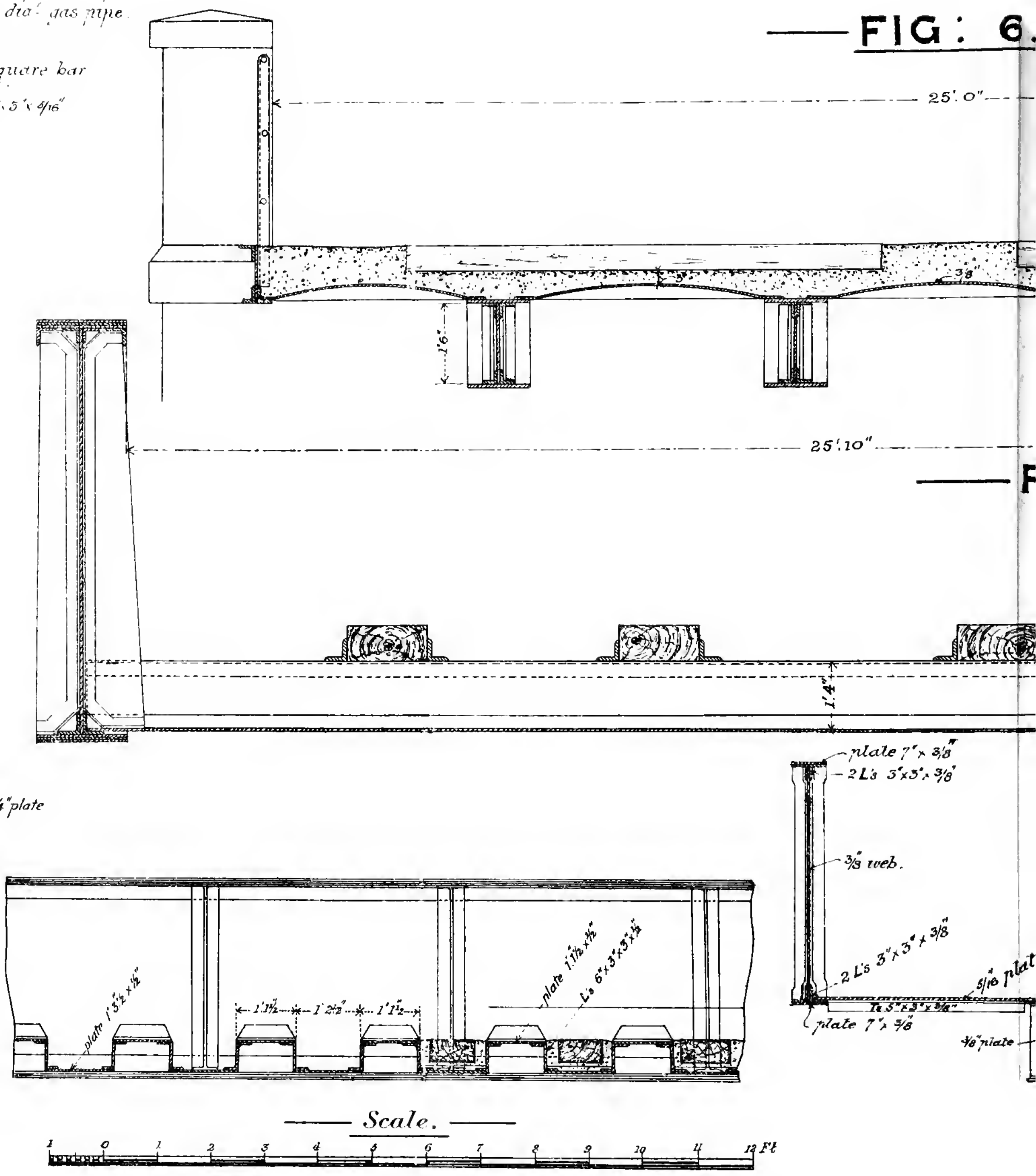

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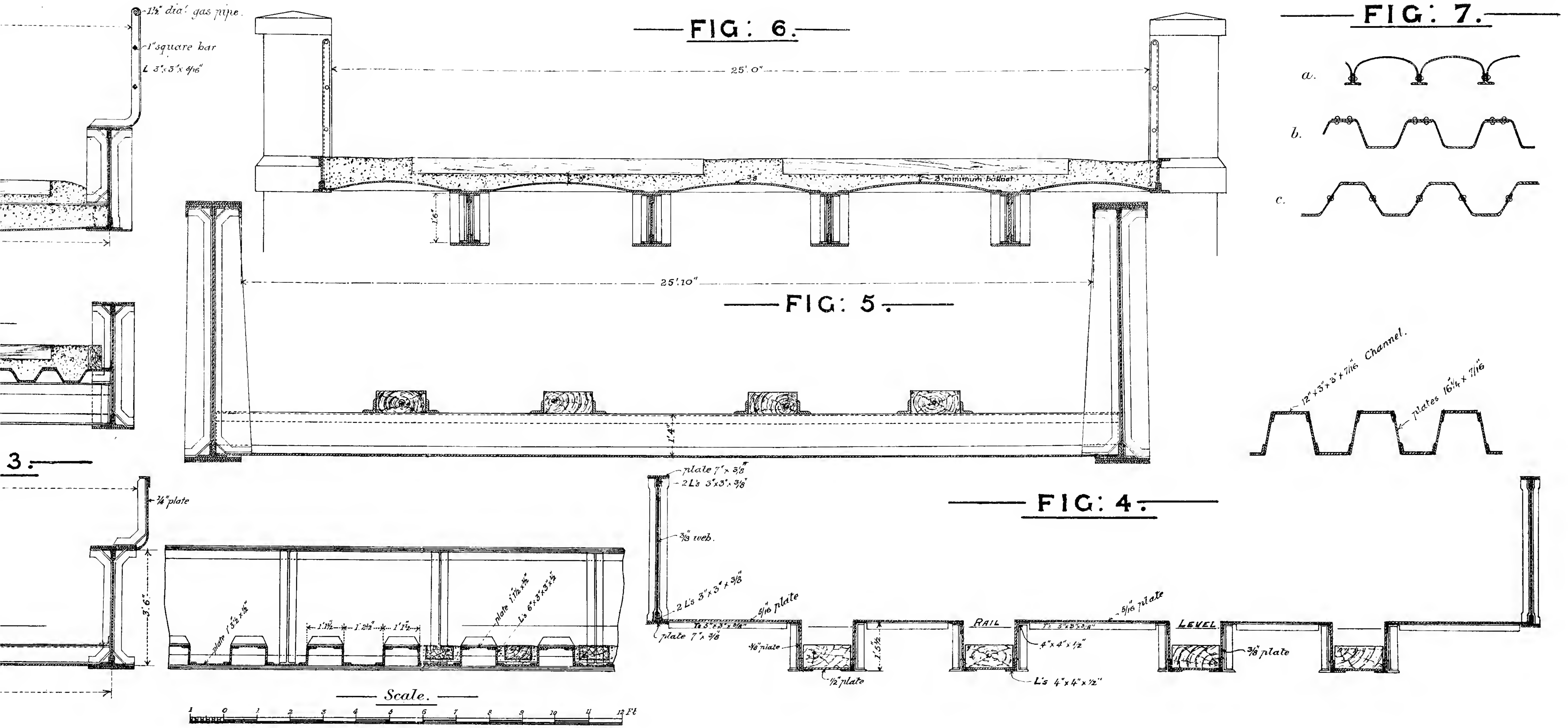


FIG: 6.
25'.0"
3"
3" minimum ballast
1'6"
FIG: 5.
25'.10"
1'.4"
FIG: 7.
a.
b.
c.
12"×3"×3"×7/16 Channel.
plates 16¼×7/16
1½" dia. gas pipe.
1" square bar
L 3"×3"×9/16"
3.
¾" plate
3'.6"
plate 1'3½×½
1'.1½
1'.2½"
1'.1½
plate 1'.1½×½
L's 6"×3"×3½
Scale.
1 0 1 2 3 4 5 6 7 8 9 10 11 12 Ft
plate 7"×3/8"
2 L's 3"×3"×3/8"
3/8" web.
2 L's 3"×3"×3/8
T's 5"×3"×3/8"
5/16" plate
plate 7"×3/8
3/8" plate
½" plate
1'.5½
RAIL
4"×4"×½"
L's 4"×4"×½"
LEVEL
5/16" plate
3/8" plate
FIG: 4.

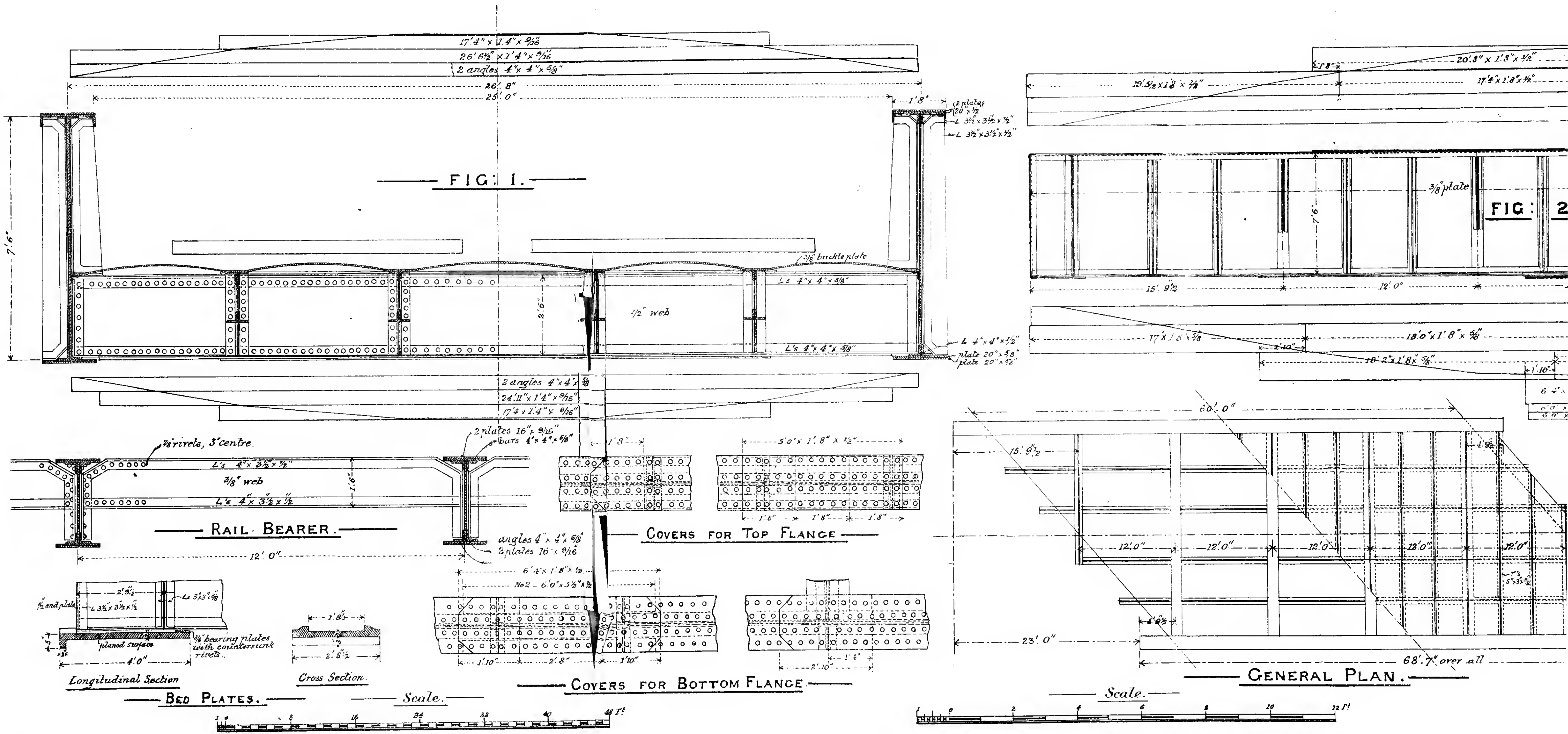
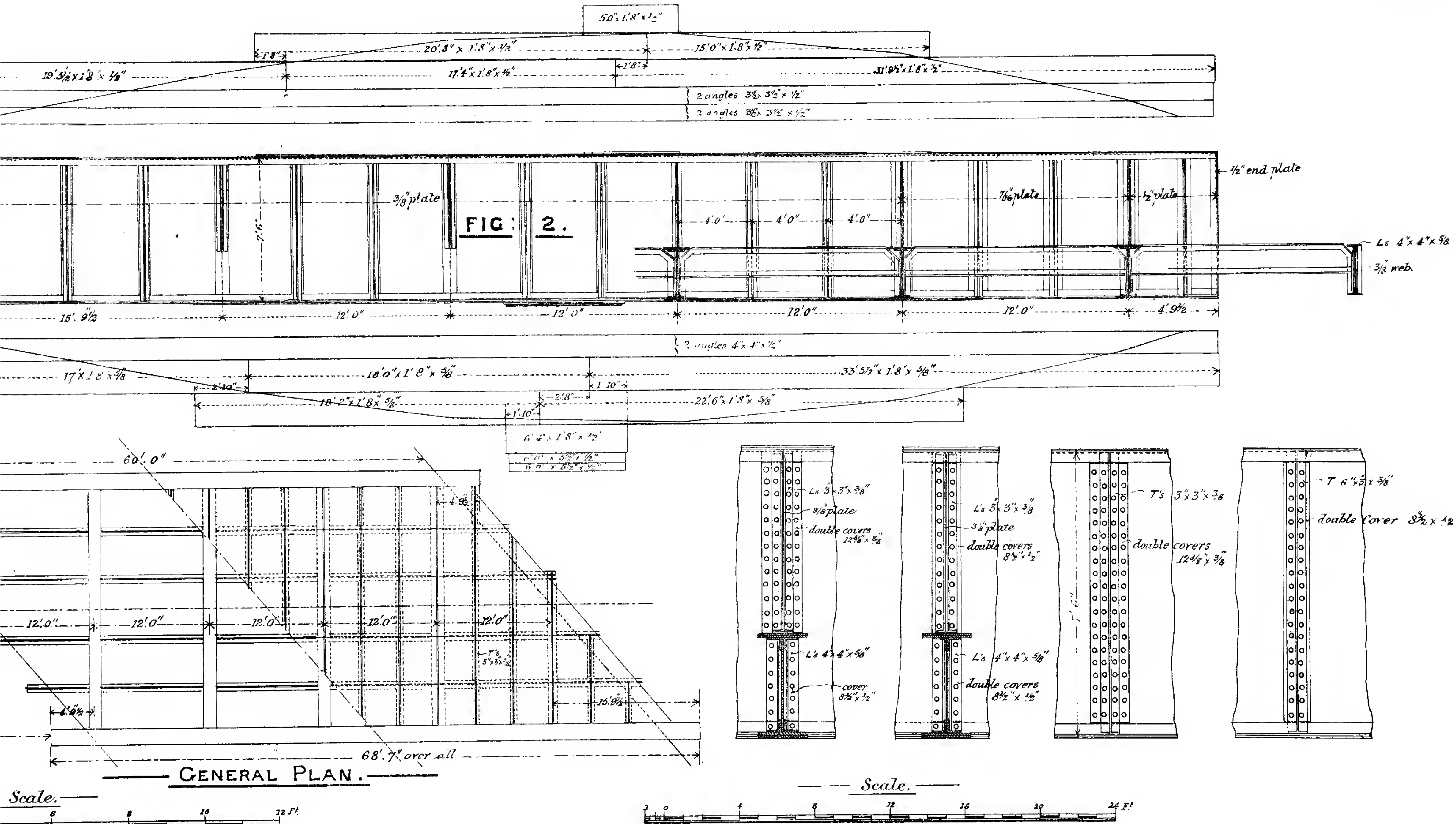


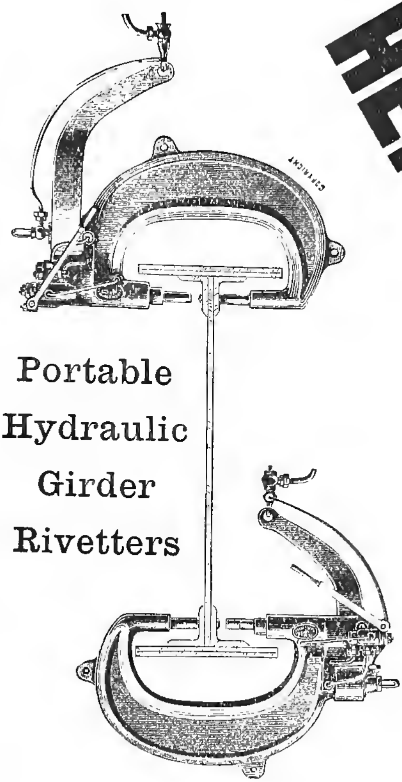
FIG: 1.
17' 4" x 1' 4" x 9/16"
26' 6½" x 1' 4" x 9/16"
2 angles 4" x 4" x 5/8"
26' 8"
25' 0"
1' 8"
7' 6"
3/8" buckle plate
L's 4" x 4" x 5/8"
½" web
2' 6"
2 angles 4" x 4" x 5/8"
24' 11" x 1' 4" x 9/16"
17' 4" x 1' 4" x 9/16"
⅞ rivets, 3" centre.
2 plates 16" x 9/16"
bars 4" x 4" x 5/8"
3/8" web
1' 6"
RAIL BEARER.
angles 4" x 4" x 5/8"
2 plates 16" x 9/16"
12' 0"
COVERS FOR TOP FLANGE
5' 0" x 1' 8" x ½"
COVERS FOR BOTTOM FLANGE
6' 4" x 1' 8" x ½
1' 10"
2' 8"
2' 10"
½ end plate
¾ bearing plates with countersunk rivets
planed surface
4' 0"
Longitudinal Section
Cross Section
2' 6½"
BED PLATES.
Scale.
FIG: 2
20' 3" x 1' 8" x ½"
17' 4" x 1' 8" x ½"
3/8" plate
15' 9½"
12' 0"
18' 0" x 1' 8" x 5/8"
17' 4" x 1' 8" x 5/8"
60' 0"
23' 0"
68' 7" over all
GENERAL PLAN.
Scale.



GENERAL PLAN.

Scale.

Scale.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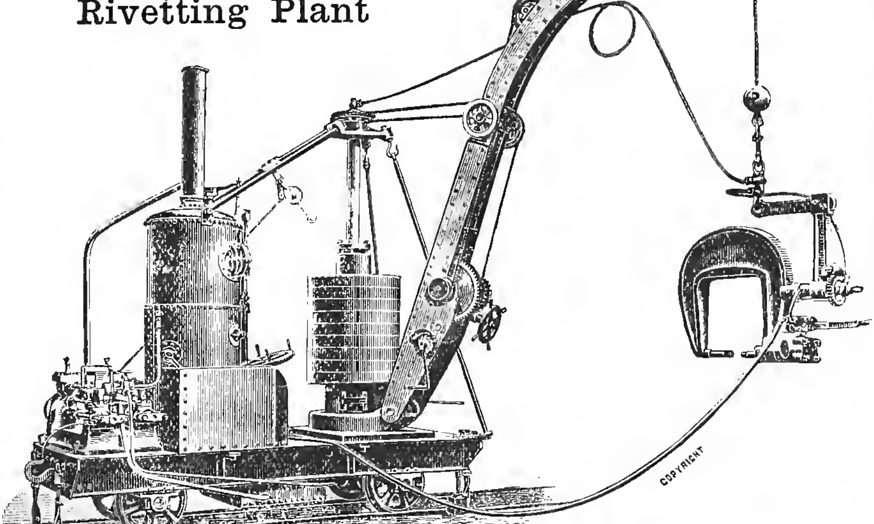
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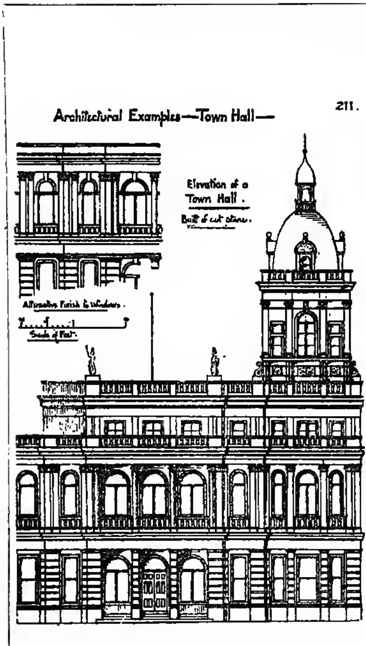

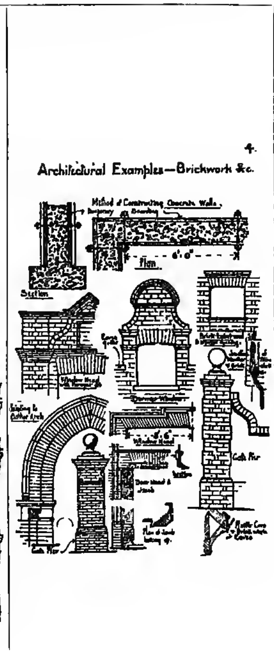

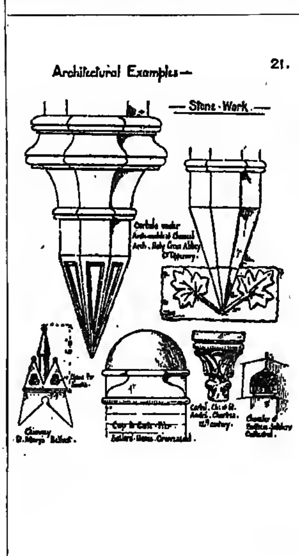

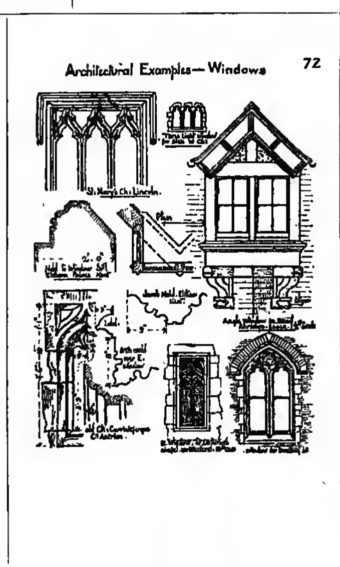

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